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FLOOD DISTRIBUTION PROBLEMS BELOW OLD RIVER

By Frederic M. Chatry<sup>1</sup>

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SYNOPSIS

Assuring the safe passage of flood flows on the Mississippi River at and below Old River requires that a controlled system of natural streams and artificial floodways be operated to distribute peak flows in accordance with the design capacities of the various system segments. Factors influencing the operation include stages and discharges, sedimentation problems, economic and commercial considerations, and adaptation to constantly changing hydraulic conditions.

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INTRODUCTION

If the limits of the watershed tributary to the Mississippi River are drawn on an ordinary map, the figure inscribed can be visualized as a large, upright funnel, with its mouth stretching across much of the northern United States, and its spout in Louisiana. In this funnel the runoff is collected from a drainage basin 1,246,000 sq miles in area, in which are contained all or parts of 31 of the states of the United States, and 2 Canadian provinces. Through the funnel's spout pour 468,000,000 acre-ft of water in an average year, at rates ranging from 100,000 cfs during low water season, to nearly 2,000,000 cfs in a normal high water, and to 3,000,000 cfs for the design flood.

When the Flood Control Act of 1928 projected the federal government into the field of flood control in a preemptive manner, the mechanism which the

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end of our funnel represents was relatively simple. Flow entered the spout via the Mississippi and Red Rivers, which streams are connected together by a short length of channel called Old River, and was then conveyed to the Gulf of Mexico by way of the Mississippi River, and the Atchafalaya River. Except for levees, neither outlet was subject to any control, and flood management was a matter of constructing and maintaining the levee system to sufficient height and cross-section to insure its integrity. However, this was not easily done, and levee overtoppings and failures were experienced in nearly every significant high water. Before embarking on a description of the control system that grew out of the Flood Control Act of 1928, some remarks on the general topography and hydrography at and below the latitude of Old River are in order. Present details of the area and of the flood control plan are shown on Fig. 1. Fig. 2 shows the area prior to start of construction of the flood control plan.

#### NATURAL SEGMENTS OF THE FLOOD CARRYING SYSTEM AT AND BELOW OLD RIVER

The largest segment of the natural complex at and below Old River is the Mississippi. The Mississippi River adopted its present course below Old River approximately 1200 A.D., after abandoning the Teche and Lafourche distributaries. The present distance from Old River to the Gulf, via Southwest Pass, is 321 miles. Below the gap at Old River, the Mississippi is fenced off to the west by a continuous levee extending almost to the river's mouth. On the east bank, the river is confined by high bluffs as far south as Baton Rouge, and by a continuous levee thereafter. The carrying capacity of the leveed channel varies throughout its length. Just above Old River there is capacity for 2,700,000 cfs. This figure decreases to 2,100,000 cfs below Old River, to approximately 1,550,000 cfs below Morganza Floodway, and finally to 1,250,000 cfs below Bonnet Carre Spillway. The Mississippi River disposes of slightly less than 70% of the total annual flow arriving at the latitude of Old River. Although this percentage has been decreasing since 1855, exhaustive studies made in connection with the Old River control project do not reveal any significant corresponding decrease in carrying capacity. As a matter of fact, the flow capacity of the River below Old River, as demonstrated by the stage-discharge relationship at that point, has been subject to no progressive change at all, but has rather been subject to both increases and decreases in flow capacity. In 1950, for example, the stage-discharge relationship was as favorable as it has ever been throughout the period for which measurements are available.

The second segment, with regard to size, is the Atchafalaya River. This stream probably was spawned by excess water from the Mississippi River spilling over the ridges of that stream's abandoned Lafourche and Teche courses. This excess water flowed into the low area, which is now called the Atchafalaya Basin and, in the process, developed a large number of small distributary basins, which ultimately joined to form the Atchafalaya River. Subsequent development of the channel was retarded by the formation of an extensive dam of drift and debris called a "raft." This raft at its point of maximum development extended downstream for approximately 30 mi and stretched from bank to bank. Pressures generated by the need for a navigation route,

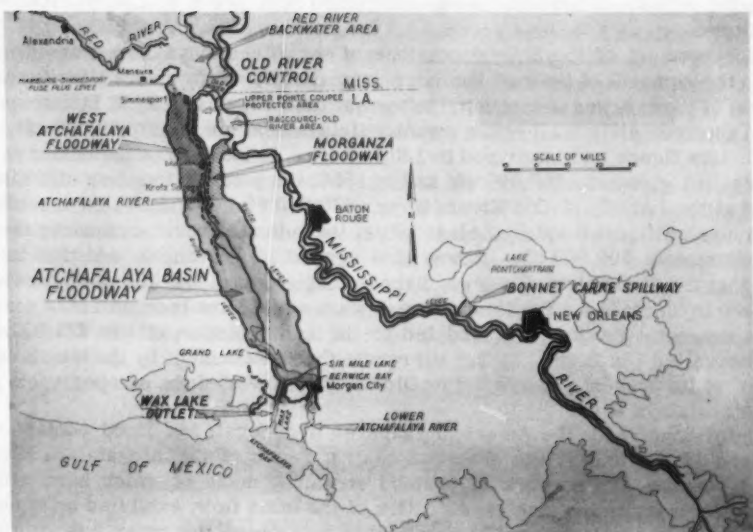


FIG. 1.—FLOOD CONTROL PLAN BELOW OLD RIVER

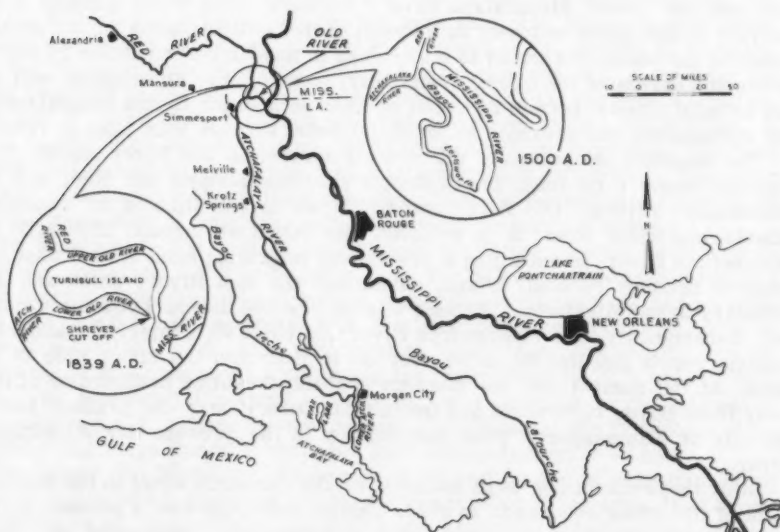


FIG. 2.—ALLUVIAL AND DELTAIC PLANES

combined with overflow of cleared lands above the raft led to its removal, undertaken in 1839. The removal was completed just prior to the War between the States.

Enlargement of the leveed portions of the Atchafalaya River during and after the removal of the raft has been dramatically swift. At the time that removal of the raft was undertaken the normal discharge capacity at Simmesport, La. (approximately 5 mi below the Atchafalaya head) was only 30,000 cfs. By 1858, this figure had increased to 130,000 cfs. At the turn of the century, the figure had grown to 260,000 cfs and in 1950, the year of the last significant flood at the latitude of Old River, it was 470,000 cfs. Even though there has been no significant flood in the last 10 yr, the indicated normal capacity is now approximately 500,000 cfs. It was this record of growth, in addition to the fact that the Atchafalaya River gradient is double that of the Mississippi, which threw into sharp focus the imminence of what would have been an act of piracy of monumental proportions, and led to the authorization of the \$70,000,000 project called Old River Control to prevent the abandonment by the Mississippi River of its present channel below Old River in favor of the steeper route via the Atchafalaya River.

Unfortunately for the American taxpayer in general, and flood control engineers in particular, the unleveed central reach of the Atchafalaya River, which consists of a network of braided streams, many of which have alignments contrary to the general direction of the main flow, exhibited no tendency toward self-enlargement. Instead, this portion of the river has continuously deteriorated through extensive siltation. The result of this siltation has been doubly detrimental to flow, not only reducing the available cross-section, but converting vast open water areas of low roughness to vegetated land areas of high roughness.

The Atchafalaya River terminates in a deep, well-defined outlet via Berwick Bay and the Lower Atchafalaya River. Because of the small quantity of silt carried in this lower segment as a result of the sedimentation in the central reach of the basin, this outlet is maintained in an efficient condition by the ingress and egress of the tides. Eventually, however, the Atchafalaya will begin to build a delta through its outlet section. In addition to this natural outlet, the Atchafalaya was provided an artificial outlet through Wax Lake in 1942.

The smallest component of the overall system is Old River itself. This stream, some 7 mi long, interconnects the Mississippi, the Red, and the Atchafalaya Rivers. Old River actually flows in one limb of an abandoned Mississippi River bend. It is believed that sometime around 1500 A.D. the Mississippi River, in enlarging a westwardly migrating bend around what was later to become Turnbull Island, caved into the Red River, which was then tributary to the Mississippi through what is now the Bayou Lettsworth channel. Subsequently, the Mississippi River continued its westerly meandering, with the result that Red River was forced to flow into the upper side of the bend. At the same time, the junction probably resulted in diversion of flow away from Bayou Lettsworth and into a distributary near the present site of the city of Simmesport. This distributary is the present day Atchafalaya River.

After these conditions were established, the climactic event in the development of Old River occurred. In 1831, Captain Henry Shreve, a pioneer in the development of waterborne commerce in Louisiana, constructed the cutoff which bears his name. Shreve's cutoff severed the migrating bend previously referred to and formed Turnbull Island. The upper portion of the bend came

to be known as Upper Old River, whereas the lower portion was called Lower Old River. Shreve's cutoff also halted the growth of the Atchafalaya River raft. Upper Old River no longer exists, except as a series of swales. Though the development of Old River lagged that of the Atchafalaya for many years, it is now developing at a rate comparable to that of the Atchafalaya River. It is probable that Old River is still inhibiting the growth of the Atchafalaya, inasmuch as it must continue to enlarge to meet the demands for water imposed by the Atchafalaya River at all stages.

The final segment of the natural system is the vast reservoir known as the Red River Backwater Area. This is the area subject to inundation by Mississippi River backwater through the gap in the levee system at Old River. Its limits embrace a total of 2,700 sq mi, and, at the elevation of the project design flood, it is capable of storing 20,000,000 acre-ft of floodwaters. Because it is subject to frequent overflow, the entire area is sparsely developed and thinly inhabited. There is, however, a significant aggregate of improved lands therein, some of which are now partially protected by levees, which makes the area a factor to be considered in distributing major floods.

#### OVERALL SYSTEM FOR CONTROL OF FLOODS AT AND BELOW OLD RIVER

In addition to the natural segments, the overall project includes several artificial segments that will be used in combination with the natural segments to distribute flood flows properly throughout the system.

The Atchafalaya Basin Floodway was formed by constructing "guide" levees east and west of the Atchafalaya River, and generally parallel to the alignment of that stream. On the east side, the levee is carried upstream to approximately the latitude of Melville, La., and then brought northeastward to join with the Mississippi River levee. On the west side, the levee extends above the latitude of Old River and ties to high ground in the vicinity of Mansura, La. The floodway thus created is intended to provide one-half of the total capacity required to dispose of the project design flood. During such a flood the peak rate of inflow to the floodway would rise to as much as 1,500,000 cfs. In order to introduce this total flow, three intakes will be employed. The first of these is the Atchafalaya River, which is not subject to control. The other two are artificial overbank floodways that flank the Atchafalaya River to the east and west, both of which are subject to some measure of control.

The floodway to the east of the Atchafalaya River, known as Morganza Floodway, receives its flow directly from the Mississippi River through a gated control structure. The structure is of reinforced concrete and consists of a weir 3,906 ft long equipped with vertical lift gates. The weir, with its crest at elevation 37.5 ft mean sea level is subdivided horizontally into 125 bays, each of which provides a clear opening approximately 28 ft wide by 25 ft deep. The Morganza Floodway extends as far southward as the latitude of Krotz Springs, La., where it will deliver its outflows to the floodplain of the central Atchafalaya Basin. It averages approximately 5 mi in width.

The intake for Morganza Floodway is located on the Mississippi River about 17 miles below Old River, in the Raccourci-Old River area. Raccourci-Old River is an ox-bow lake, formed by the Raccourci cutoff which occurred in 1848. The lake is expected to materially assist ingress of floodwaters into the floodway, and is further expected to act as a natural silt basin after the floodway is placed in use.



The Morganza Floodway is designated to divert approximately 600,000 cfs from the Mississippi River to the Atchafalaya Basin. The lift gates provide complete flexibility in operation, in that flow through the floodway can be started, stopped, or adjusted as required. The Morganza Floodway was completed in 1956, and has not been used since completion. An overall view of the control structure is shown in Fig. 3. Details of the weir and gates can be seen in Fig. 4.

On the western side of the Atchafalaya River lies the West Atchafalaya Floodway, larger in area than the Morganza Floodway and potentially much more controversial in operation. The West Atchafalaya Floodway differs from the Morganza Floodway in that control is not achieved by the positive mechanism of gates, but by a levee of substandard grade called a fuseplug levee. In theory, the grade of such a levee is set to permit overtopping when the flood in progress reaches the point where operation of the floodway is required to obtain the desired flow distribution. After overtopping, natural erosion of the levee, augmented as required by artificial degrading will be relied upon to insure the required diversion.

The West Atchafalaya Floodway extends southward to the latitude of Krotz Springs where, like the Morganza Floodway, it will discharge its floodwaters to the central Atchafalaya Basin Floodway. It averages approximately 8 mi in width. Unlike Morganza Floodway, which is practically uninhabited, the West Atchafalaya Floodway supports a population of approximately 3,000 and contains numerous improvements and much livestock.

The West Atchafalaya Floodway has not been used, and will only come into operation for a flood which approaches the project design value in magnitude. Fig. 5 shows the fuseplug levee at the head of the floodway.

The final component of the Atchafalaya Basin Floodway is the Wax Lake Outlet into the Gulf of Mexico. This auxiliary outlet was constructed in 1942 by dredging a channel from Six Mile Lake, through the Bayou Teche Ridge, to Atchafalaya Bay. The auxiliary outlet accommodates approximately 20% of the total Atchafalaya Basin flow. An aerial view of the outlet is shown in Fig. 6.

The third artificial floodway in the overall plan is Bonnet Carre Spillway, which is located on the east bank of the Mississippi River 33 miles above New Orleans. It was constructed to divert excess Mississippi floodwaters into Lake Pontchartrain. It is the oldest of the artificial floodways, having been completed in 1936. The control structure, which is located at the river end of the Floodway, consists of a reinforced concrete weir, 7,698 ft in length, subdivided into 350 bays, 176 of which have a crest elevation of 15.8 ft msl, and 174 of which have a crest elevation of 17.8 ft msl. Control is by wooden timbers or "needles" which measure 8 in by 12 in. The Spillway was designed to discharge a peak flow of 250,000 cfs at a stage of 20 ft at New Orleans. It has been operated in 1937, 1945, and 1950. In 1945, the design discharge was equaled or exceeded on 15 days.

Significant problems have been encountered in the operation of Bonnet Carre Spillway, in that sedimentation in both the forebay area (the area between the river bank and the control weir) and the Spillway proper are operating to reduce its capacity in a progressive manner. Although the flow capacity can be maintained indefinitely by removing the silt deposits, such maintenance is very expensive. View of the Spillway in operation are shown in Figs. 7 and 8.

Although it is not yet a functioning segment of the overall plan, the Old River Control project is sufficiently close to completion that consideration of it should properly be included in any description of the overall system.



FIG. 3.—CONTROL STRUCTURE OF MORGANZA FLOODWAY. MISSISSIPPI RIVER VISIBLE IN THE BACKGROUND TO THE LEFT.

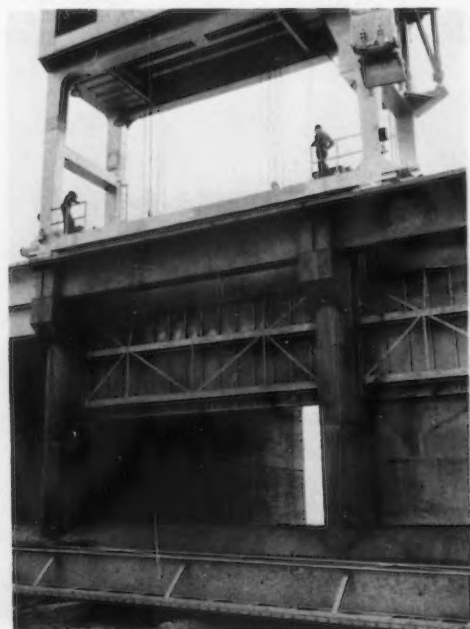


FIG. 4.—MORGANZA FLOODWAY. CLOSE-UP VIEW OF CONTROL STRUCTURE, FROM FOREBAY.





FIG. 5.—WEST ATCHAFALAYA FLOODWAY. SIMMESPORT-HAMBURG LEVEE AT HEAD OF THE FLOODWAY.



FIG. 6.—WAX LAKE OUTLET. VIEWED IN THE DIRECTION OF FLOW.

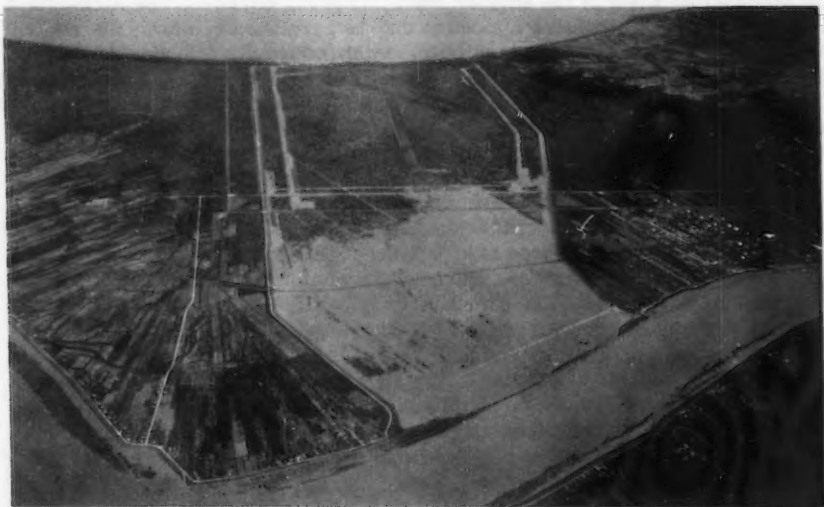


FIG. 7.—BONNET CARRE SPILLWAY. VIEW DURING 1950 OPERATION. MISSISSIPPI RIVER IN FOREGROUND. LAKE PONTCHARTRAIN IN BACKGROUND.



FIG. 8.—BONNET CARRE SPILLWAY. CLOSE-UP OF CONTROL STRUCTURE 1950 OPERATION. ONE BAY PARTIALLY OPEN.

This project is one of the largest attempts at flood control engineering undertaken in the United States. Its basic purpose is to prevent the adoption by the Mississippi River of the shorter route to the sea offered by the Atchafalaya River. The project includes a closure dike to permanently sever the natural connection between the Mississippi and Atchafalaya Rivers via Old River, a gated control structure with inlet and outlet channels to provide for necessary flow diversions from the Mississippi to the Atchafalaya under all conditions, and a navigation lock to preserve the navigational opportunities now afforded by Old River.

The control structure consists of two independent units in order that the necessary flexibility in operation may be assured. The so-called low sill portion consists of eleven openings, 44 ft wide. Of these eleven openings, eight are 55 ft deep and three are 70 ft deep. All openings are controlled by multiple-leaf vertical lift gates, which will be manipulated to control low water and medium stage flow. The overbank or high sill structure consisting of 73 bays, 44 ft wide and approximately 12 ft deep, equipped with needle type closures, will control diversion of high water and major flood flows. The two structures are capable of passing a flow of approximately 700,000 cfs under existing channel conditions in the Mississippi and Atchafalaya Rivers. Continued development of the Atchafalaya River will increase this maximum capacity.

At the present time, the control structure, the outflow channel, and most of the levees are complete. The navigation lock is under construction, and the entire project is scheduled for completion in 1964. However, should an emergency develop in which the need for closing Old River became immediate, the control structure could be operated.

In general, Old River Control will be operated in such a fashion as to distribute the flows artificially in almost the same proportions in which they would have been distributed under the natural conditions that existed in 1950. The details of operation are not appropriate for presentation herein; however, the entire operation will be oriented around the proposition of preserving the existing stability of the Mississippi River channel below Old River. An overall aerial view of the salient project features is shown in Fig. 9.

#### OPERATION OF THE VARIOUS FLOODWAYS FOR DISTRIBUTING FLOOD FLOWS AT AND BELOW OLD RIVER

The natural elements of the flood control plan below Old River are self-operating. The artificial floodways and control structures are operated in such a manner as to insure that the safe capacities of the various segments of the system are not exceeded. In the operation, guide lines are provided by stages and discharges. There is only one specific operational criterion with the sanction of law, and it is that the Bonnet Carre Spillway shall be operated to prevent stages at New Orleans from exceeding 20 ft.

The natural leveed channel of the Mississippi River at and below New Orleans can accommodate all floods in which the peak flow in that reach of the river does not exceed 1,250,000 cfs. A flood producing this peak flow can be expected to occur on an average of once every 5 to 7 yr, and would produce a maximum stage at New Orleans of 20 ft. Such a flood represents the threshold flood insofar as floodway operation is concerned.

For floods exceeding the magnitude of that previously described, operation of one or more of the artificial floodways is required. In a typical flood the sequence of operations would generally be as follows.

1. Operation of either Bonnet Carre Spillway or Morganza Floodway when a flow in excess of 1,250,000 cfs is in sight for the Mississippi River below Old River.
2. Operation of the remaining Mississippi River floodway when a flow in excess of approximately 1,500,000 cfs is in sight for the Mississippi River below Old River.
3. Operation of the West Atchafalaya Floodway when the total flow at the latitude of Old River exceeds approximately 2,800,000 cfs.

#### OPERATIONAL PROBLEMS

If the relationships between stage and discharge throughout the various segments of the flood control system below Old River were constant, the operational procedure would be straightforward and invariable. Whereas flow in



FIG. 9.—OLD RIVER CONTROL. LOW-SILL CONTROL STRUCTURE AND OUTLET CHANNEL IN CENTER OF VIEW. HIGH-SILL CONTROL STRUCTURE TO THE RIGHT.

alluvial rivers follows the same immutable laws governing all fluid flow, physical changes in the hydrography of these streams produce continuing changes in their hydraulic efficiency. In addition to well-defined changes of a progressive nature, such as those taking place on the leveed portion of the Atchafalaya River, other changes for which cause-and-effect relationships have not been established combine with the aforementioned, and operate to make an inflexible operational procedure impracticable. Notable among these is the variation in stage-discharge relationships for the Mississippi River itself.

The intake capacities of all the artificial floodways are affected by the changes previously mentioned. The Morganza Floodway intake capacity is indicated to be in the neighborhood of 50,000 cfs per ft of stage for project design conditions. Inasmuch as the stage-discharge relationship for the Mississippi River below Old River has varied by as much as 2 ft for large flows, the extent to which these variations influence diversions through Morganza Floodway can be readily appreciated. The significance of these variations is enhanced when it is considered that they tend to vary from one section of the river to another. For example, the stage discharge relationship in the Mississippi River between Old River and Baton Rouge was, during the 1950 flood, as low as it had been for the entire period of record, whereas such was not the case below Baton Rouge.

The performance of the West Atchafalaya Floodway is influenced by variations in stage-discharge relationships to an even greater extent. For example, as recently as 1941, the planned distribution of the project design flood anticipated simultaneous flows of 650,000 cfs and 250,000 cfs, in the Atchafalaya River and the West Atchafalaya Floodway, respectively. In 1950 the stage along the fuseplug levee at the head of the floodway averaged 5.5 ft below the fuseplug levee grade even though the peak flow in the Atchafalaya River reached 650,000 cfs. The intake difficulty imposed by the steady lowering of the stage-discharge relationship for the Atchafalaya River at Simmesport, La. will be worsened as time goes by.

The general problem of properly distributing floods below Old River involves providing answers to the following questions.

1. At what time during the flood should the first floodway be operated?
2. Which floodway should be operated first?
3. At what time during the flood should the second floodway be operated?
4. If the flood is indicated to be of extreme magnitude, what steps should be taken to insure that the West Atchafalaya Floodway operates in a timely and effective manner?

*Time of Initial Operation.*—This problem involves careful consideration of antecedent stages and discharges on the Mississippi River. Because project stages will be approached along the entire Mississippi below Bonnet Carre Spillway prior to the operation of the first floodway, whereas Atchafalaya Basin stages will be relatively moderate at that time, control and limitation of stages in that reach or river are the primary concern. Accordingly, the operation must be closely coordinated with the stage at New Orleans. Ordinarily, a fairly definite decision can be reached as to the proper timing of the initial operation. Although unusual, cases will arise in which the proper time of initial operation will be somewhat obscure. This could happen in a flood cresting at a discharge slightly larger than 1,250,000 cfs in the Mississippi River, particularly if the flood hydrograph rose rapidly. Such a flood would probably produce unusually low stage-discharge relationships on the Mississippi River in the early phases of the flood, in which case there would be a tendency to delay the operation in the hope of avoiding it altogether. This procedure could result in waiting too long, and thereby permitting the New Orleans stage to go above 20 ft. On the other hand, early operation based on discharges alone, might result in an operation that could otherwise be avoided.

No all-inclusive specific rule can be laid down for time of initial operation, but, in general, it would appear proper to base a Morganza first operation on Mississippi River discharge, and a Bonne Carre first operation on New Orleans



stage. Thus, if Morganza Floodway were to be operated first, this operation would come sufficiently in advance of the Mississippi River discharge reaching 1,250,000 cfs to insure that not more than 1,200,000 cfs would flow past New Orleans. If Bonnet Carre Spillway were to be operated first, the operation would come at a stage of approximately 19+ ft at New Orleans, with more than 20 ft in prospect.

*Which Floodway to Operate First.*—This question is a great deal more complex than that of when to operate. There are advantages and disadvantages to both operational sequences. The operation of Bonnet Carre Spillway is physically quite simple. The land on which the spillway is located is owned in fee simple by the federal government, and the problem of evacuation is nil. The Bonnet Carre operation involves little or no economic or commercial disruption. Although unanimity of opinion does not exist with respect to the effect of a spillway operation on fish and wildlife opportunities, most informed experts agree that operation of the spillway is helpful in this regard. Based on experienced flows in Bonnet Carre Spillway and theoretical computations and model studies for Morganza Floodway, it is indicated that the magnitude of the flood that the overall system can safely accommodate with only Bonnet Carre Spillway in operation is greater than that which would be accommodated with only Morganza Floodway. In practical terms, this means that floods cresting near 1,500,000 cfs in the Mississippi below Old River could be handled with Bonnet Carre alone without exceeding 20 ft at New Orleans, but not with Morganza alone.

The most serious disadvantage of a Bonnet Carre operation lies in the sedimentation concomitant to operation. In addition, the stage lowerings realized from a Bonnet Carre operation are limited essentially to the Mississippi River below the Spillway itself.

The Morganza Floodway, on the other hand, provides stage lowerings and benefits in the Red River backwater area as well. Based on present best estimates, the sedimentation problem in Morganza Floodway will be much less serious than that in Bonnet Carre Spillway.

On the unfavorable side, operation of the Morganza Floodway is much more complex than that of Bonnet Carre Spillway, and its effects will be felt over a greater area. There is a substantial evacuation problem involved. Backwater flooding within the West Atchafalaya Floodway Area will be increased. Drainage in the Pointe Coupee Protected Area, and in the towns of Melville and Krotz Springs, will be interrupted or impaired, and although dedicated areas for ponding runoff have been provided, there is always the outside chance that an extreme rainfall will overtax the storage provided.

On an overall basis, it would appear that the advantage lies with a Bonnet Carre first operation. However, it must be borne in mind that sedimentation may well be a controlling element. In any event, it is planned to operate the Morganza Floodway first in the next flood requiring operation of a floodway, in order that the definite information needed to calibrate the control structure, determine overall hydraulic effects, and evaluate the sedimentation problem in a quantitative manner may be collected.

*Time of Operation of Second Floodway.*—The factors governing this problem are similar to those relating to time of operation of the first, except that the threshold discharge will be approximately 1,500,000 cfs. As before, if Morganza is to be operated, the operation would be keyed generally to the discharge, whereas a Bonnet Carre operation would follow more closely the New Orleans stage.

*Operation of West Atchafalaya Floodway.*—Although the West Atchafalaya Floodway is not subject to control in the same sense that Morganza Floodway and Bonnet Carre Spillway are, it nonetheless presents a most complex operational problem. The central element in this problem is the leveed channel of the Atchafalaya River. As mentioned previously, the operation of the West Floodway is automatic and will result from the stage along its fuseplug levee exceeding the levee grade. The natural enlargement of the Atchafalaya River has progressively lowered the stage along the fuseplug for a given total flow at the latitude of Old River. Although such a development may be viewed as generally favorable, in that it is producing a continuous increase in the overall intake capacity at the head of the Atchafalaya Basin, it does introduce an impediment to timely operation of the West Atchafalaya Floodway, because such operation must await the natural overtopping of the fuseplug. Failure of the West Atchafalaya Floodway to operate on time would probably result in overtaxing the safe leveed capacity of the Mississippi River, particularly below New Orleans.

At the present time, it appears that operation of the West Atchafalaya Floodway will commence early enough in a great flood to limit Mississippi River flows to a safe value, provided that the initial overtopping of the fuseplug is followed promptly by various measures designed to facilitate entry of floodwaters into the Floodway. Such measures might include degrading of the fuseplug levee at the head of the floodway or its destruction by explosives. Similar action may be taken along the West Atchafalaya River levee, if required. However breaching the West Atchafalaya River Levee should be avoided, if possible, because it would produce excessive velocities in the Atchafalaya River above the breach, and could result in uncontrolled bank caving and loss of the bridge across the Atchafalaya River at Simmesport.

In view of the fact that continued improvement of the Atchafalaya River is expected, the West Atchafalaya Floodway problem will tend to grow progressively worse. The long-term solution lies in revision of the grades of levees protecting the floodway.

### CONCLUSIONS

Any plan for the control of floods below Old River must possess the following attributes:

1. It must be capable of safely distributing the maximum peak flows that may reasonably be expected to occur.
2. It must accomplish this distribution with a minimum of economic disruption.
3. It must be adaptable to changing conditions, so that its effectiveness may remain unimpaired in the future.

The system described herein is shown to satisfy these requirements. It can be operated in a planned and predictable manner for peak total flows at the latitude of Old River up to and including 3,030,000 cfs, a value that has been established by extensive meteorological and hydrological studies as the largest likely to occur. With the exception of the West Atchafalaya Floodway, which will operate less often than once every hundred years, the economic disruption deriving from system operation is not great. It varies from negligible in the case of an operation of Bonnet Carre Spillway to minor for a Morganza



Floodway operation. And the overall system's flexibility and adaptability are such that the end of its useful life cannot now be foreseen. With the system operating, the threat of damaging overflow below Old River has been virtually eliminated.

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PREDICTING STORM RUNOFF ON SMALL EXPERIMENTAL WATERSHEDS

By Neal E. Minshall,<sup>1</sup> M. ASCE

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SYNOPSIS

Careful estimates of rates and amounts of runoff are required for design of flood retarding and channel stabilizing structures, and bridges and culverts in small upstream watersheds. The period of record of many of the small experimental watersheds in the United States is too short to permit direct analysis for reliable estimates of the magnitude and frequency of storm runoff. A method is presented for extending the period of runoff records based on analysis of existing short term records of rainfall and runoff for the watershed, and a longer record of the rainfall alone. The method involves: (1) estimating storm runoff volumes from the rainfall pattern and antecedent rainfall, and (2) distributing this runoff through an adaptation of the unit hydrograph principle. A method is also presented for developing synthetic unit hydrographs for ungaged areas.

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INTRODUCTION

A very limited amount of runoff data is available on small watersheds, and these records are not generally of sufficient length to provide reliable estimates for design floods. Since records of precipitation are generally available for much longer periods than runoff records, it would be desirable to establish precipitation-runoff relations and use such relations to develop flood

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Note.—Discussion open until January 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 8, August, 1960.

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hydrographs from the longer precipitation record. The method presented here is summarized as follows:

- (1) Develop time-retention curves for each soil cover combination, based on an antecedent precipitation index and season.
- (2) Estimate the storm runoff volume from the precipitation pattern and the time-retention curve.
- (3) Derive unit hydrographs for various rainfall intensities.
- (4) Apply these relationships to a number of outstanding rainfalls and compare the computed hydrographs with the observed runoff.

The relationships developed can be applied to a longer precipitation record from a nearby recording rain-gage. A relationship has also been developed to permit adaptation of these principles to ungaged areas.

The analysis for developing the procedures was made with rainfall and runoff data from three small experimental watersheds located near Edwardsville, Ill. (Fig. 1).

#### DESCRIPTION OF EXPERIMENTAL WATERSHEDS

The three Edwardsville watersheds, with a large number of runoff periods each year, offer an excellent opportunity for studies of rainfall-runoff relations on small watersheds. These watersheds (details shown in Fig. 2) are designated as: W-1, a 27.2-acre cultivated area; W-2, a 50-acre pastured area; and W-4, a 290-acre mixed cover watershed. The two small watersheds are part of the 290-acre area. These watersheds are, in general, quite flat in the upper portions but drop off rather abruptly near the waterways.

Watershed W-1 is a fan-shaped area with a range in elevation of 20 ft, and more than two-thirds of the area has slopes less than 1%. This area was 96% cultivated during the years of 1938 and 1948-50, 80% cultivated in 1951 and 1955, 100% alfalfa in 1940 to 1943, and 44% cultivated during the rest of the period of record. Areas not cultivated were in legume hay.

Watershed W-2 is an oval-shaped area having a length about one and one-half times its width. The difference in elevation between the divide and the runoff station is 39 ft. About one-third of the area has an average slope of 1.5%, and another third has 12% slopes. During the period of record, cover on this area has varied from 0 to 15% cultivated with the remainder in pasture.

Watershed W-4 is nearly circular and has a range in elevation of 63 ft. One-third has an average slope of 1%, and another third has 12% slopes. The maximum slope near the waterway is 35%. There are four small ponds with a total drainage area of approximately 30 acres or 10% of the watershed area. During the period of record, from 17 to 30% of this area has been cultivated and the remainder mostly in pasture.

The soils on these watersheds are of loessial origin over glacial till with an impervious claypan layer at depths of 10 to 20 in. These soils, when dry and protected with a good vegetative cover, will take in precipitation rather rapidly until the surface layer becomes saturated, after which additional precipitation will result in a high percentage of runoff. The data obtained on these areas are applicable to the central claypan areas of southern Illinois, southern Indiana, southwestern Ohio, Missouri, eastern Kansas, and northeastern Oklahoma. The mean annual precipitation over most of this area is about 40 in., but the mean monthly rainfall varies widely from a fairly uniform distribution

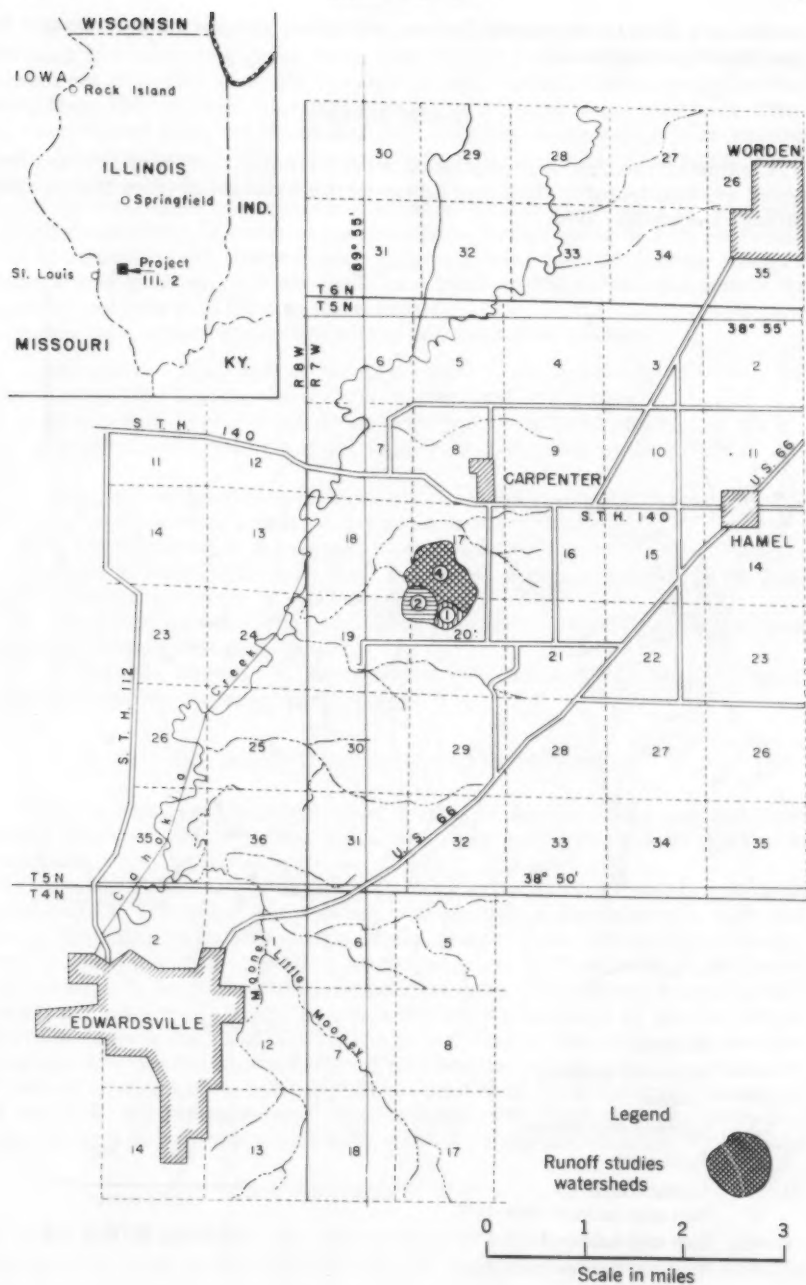


FIG. 1.—LOCATION OF WATERSHEDS

in southern Illinois, to eastern Kansas, which has high spring and summer but low winter precipitation.

### BASIC DATA

Continuous records of precipitation, runoff, temperature, humidity, and crop cover were collected on the three Edwardsville watersheds from March, 1938, through December, 1955.

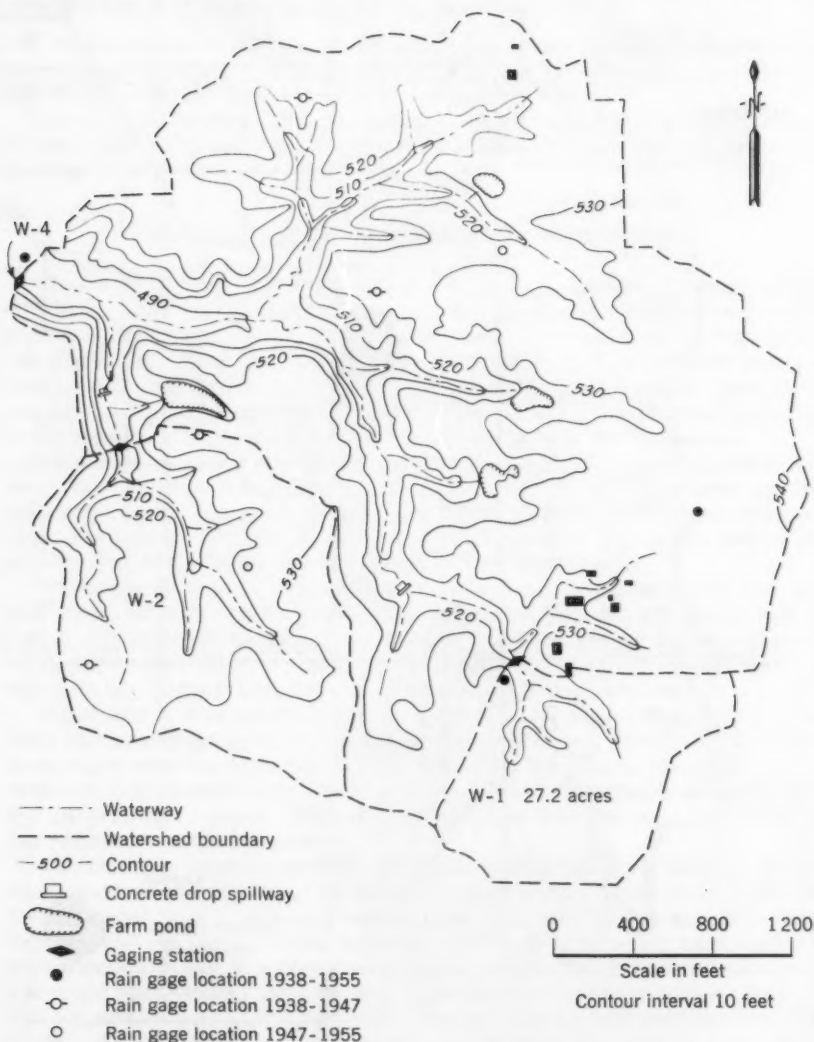


FIG. 2.—MAP OF EDWARDSVILLE, ILL., WATERSHEDS

Precipitation was measured with seven recording raingages from 1938 to 1947 and five recording gages from 1948 through 1955. Additional recording gages were in operation on the two small areas during an intensive infiltration study from 1940 to 1943. The only gage on the 27-acre area, except for 1940-43, was located near the runoff station. Because of the variation in rainfall during thundershowers, this gage may not be representative of the actual average precipitation on the area.

Runoff from these areas was measured with concrete V-notch broad crested weirs equipped with water stage recorders. Runoff records from watershed W-4 are based on the effective area varying in size from 290 acres, when all ponds are at spillway elevation prior to rainfall, to 260 acres when none of the ponds are discharging through the spillway.

*Definitions.*—The terms used herein are defined as follows:

1. Retention - That part of the precipitation not appearing as runoff, includes plant interception, depression storage, and infiltration.
2. Antecedent Precipitation Index (API) - An empirical measure of the effect of precipitation falling a given number of days prior to the period of excess.
3. Runoff - As used here includes all flow that eventually passes over the weir, including surface runoff, interflow, and base flow.
4. Unit Hydrograph - A hydrograph representing 1 in. of runoff.
5. Time to Peak - The time from beginning of excess rainfall to the peak rate of runoff for storms of short duration.
6. Period of Excess - The time during which precipitation occurred at rates in excess of the infiltration capacity.
7. Infiltration Capacity - The maximum rate at which a given soil, when in a given condition, can absorb falling rain.<sup>2</sup>

### ESTIMATING STORM RUNOFF VOLUMES

Analysis was first directed toward developing a relationship between antecedent precipitation (as a measure of the available soil moisture storage) and retention.

*Computing Antecedent Precipitation Index.*—The length of time that must be considered in computing any antecedent precipitation condition will vary with the permeability of the soils and subsoils, temperature, evaporation, transpiration, and vegetation. For very permeable soils such as sands or the better silt loams, a 5-day antecedent precipitation index may suffice, whereas, for a slowly permeable soil it may be necessary to use a period of 30 days or longer. Various methods and lengths of period were tested in computing the antecedent precipitation index for Edwardsville areas, and the one that gave the best correlation with retention was a 30-day index plus 20% of the departure from normal of the 30-60 day precipitation. The final selection which gave nearly as good relation with less computation was based on a 30-day period using the formula:

$$API = P_0 + P_1k + P_2k^2 + P_3k^3 + \dots + P_nk^n \dots \dots \dots (1)$$

in which API is the antecedent precipitation index,  $P_0$  refers to precipitation within 24 hr prior to the excessive period which produced the runoff shown in

<sup>2</sup> "Surface Runoff Phenomena, Part I—Analysis of the Hydrograph," by Robert E. Horton, Publication 101, Horton Hydrological Lab., Voorheesville, N. Y.



column 6 at Table 1,  $P_1$ ,  $P_2$ , and  $P_3$  indicate precipitation 1, 2, 3 days, and so on, prior to the storm,  $n$  denotes the number of days used to establish the index (in this case 30), and  $k$  is a constant depending primarily on the soil type (0.95 gave the best results for the claypan areas).

*Relation of Antecedent Precipitation to Retention.*—Initial investigations showed that a large volume of data was needed to develop relationships between antecedent precipitation index and retention. Therefore, consideration was given to combining the data from watersheds W-1, W-2, and W-4. Comparison of amounts of precipitation retained from the excessive portion of storms showed that retention on W-1 (27.2 acres - all alfalfa), during 1940-1943 was about 95% of that on W-4. When W-1 was 96% cultivated, the retention for individual storms averaged 18% less than W-4. Retention on W-2 (50 acres - of all pasture) averaged 5% more than on W-4. Comparison of annual and seasonal runoffs on watersheds W-1 and W-2 against W-4 show that for the period of 1944-1955, the average runoff on the two small areas was only 9% more than on W-4. With the apparent small variation between watersheds, it was felt that data from all three areas could safely be combined for the retention analysis.

Table 1 gives the data for watershed W-4 for all storms during the 18 yr of record that caused runoff in excess of 0.1 in. and where the precipitation was such that all runoff could be charged to a definite period of rainfall excess. Where it was possible to break a storm down into several distinct runoff periods, each of these periods was entered in the table. Some winter storms were eliminated because of frozen ground or melting snow. Similar information to that presented in Table 1 was compiled for watersheds W-1 and W-2.

The data for all watersheds were divided into several groups according to season and further subdivided by groupings based on the API. In grouping these data according to season, one should recognize that variation in season from one year to the next may be as much as two weeks. Because of this, it is not entirely logical to arbitrarily select a given date as the limit of a particular group. The year was first divided into spring, summer, fall, and winter with three subdivisions for API in each, giving a total of 12 groups of data.

The data from columns 4 and 6 of Table 1 and similar information from W-1 and W-2, grouped as explained previously, were plotted on rectangular coordinate paper. The results indicated a rapidly changing infiltration capacity during the spring and fall seasons and fairly constant conditions during the summer and the dormant season. They also indicated the advisability of slightly different grouping according to seasons. The final groupings, selected as giving good results with a minimum of groups, were: November 15 to April 15; April 15 to May 15; May 15 to June 30; July through September; and October 15 to November 15.

Fig. 3(a) shows these time retention curves for the period July through September at different antecedent precipitation indexes. Curves were fitted in to the groups of points as plotted. Similar sets of curves were prepared for each of the periods listed. Fig. 3(b) shows the maximum retentions that result from antecedent precipitation indexes less than 0.5. Fig. 3(c) shows retentions for the various periods, termed average because they occur more frequently than the others. To continue these curves to the origin of time and depth would serve no purpose. Those representing dry or summer conditions would break down very sharply, indicating the magnitude of plant interception, depression storage and initial infiltration rate. Minimum infiltration rates on these soils are nearly constant throughout the year varying between .02 and .06 in. per hr.

TABLE 1.—DATA FROM SELECTED STORMS CAUSING  
MORE THAN 0.1 IN. RUNOFF, 1938-55

Date	Precip. before excess Inches	Excessive Precipitation		Runoff from excess, Inches	Retention from excess, Inches	30-day Antecedent Precipita- tion Index (A.P.I.)
		Amount, Inches	Duration Hrs. Min.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Mar. 13, 1938	.04	.85	5 30	.17	.68	1.78
Mar. 13,	0	.76	2 40	.44	.32	2.59
Mar. 15,	0	.59	2 07	.28	.31	2.87
Mar. 15,	0	.45	0 28	.39	.06	3.43
Mar. 22,	0	.66	0 22	.20	.46	2.46
Mar. 28,	0	.58	3 50	.18	.40	2.54
Apr. 8,	.10	.68	1 45	.52	.16	4.95
May 27,	.04	1.30	0 32	.63	.67	2.68
June 10,	0	.71	0 08	.20	.51	2.77
June 23,	0	.68	0 55	.16	.52	2.36
July 17,	.93	3.28	2 36	1.33	1.95	1.48
Feb. 9, 1939	.53	.36	0 13	.27	.09	2.03
Feb. 19,	.03	.93	5 30	.37	.56	1.55
Mar. 11,	.15	.61	2 17	.18	.43	1.26
Mar. 11,	0	.40	1 50	.27	.13	1.86
Apr. 15,	0	.55	4 00	.33	.22	1.99
Apr. 20,	0	.43	5 00	.13	.30	2.45
July 16,	.55	1.98	1 20	.58	1.40	2.46
Aug. 11,	.08	.84	0 12	.22	.62	2.64
Aug. 11,	0	.58	0 40	.19	.39	3.39
Aug. 12,	.25	.47	0 45	.16	.31	4.38
Aug. 17,	.07	4.42	9 20	1.80	2.62	3.25
Apr. 30, 1940	.05	1.14	4 07	.35	.79	1.49
Aug. 4,	.05	3.58	2 35	.73	2.85	.19
Apr. 17, 1941	.53	.49	0 13	.16	.33	2.23
July 10,	0	1.25	0 55	.25	1.00	2.21
Sept. 2,	0	1.43	1 00	.21	1.22	1.58
Sept. 2,	0	.57	0 25	.24	.33	2.88
Oct. 9,	.07	.64	0 17	.10	.54	2.22
Oct. 22,	.04	.49	0 20	.12	.37	3.45
Oct. 22,	.59	.54	0 40	.29	.25	3.98
Nov. 5,	0	1.95	17 00	.78	1.17	3.53
May 5, 1942	.10	1.00	1 45	.19	.81	1.58
June 13,	.05	1.16	0 33	.40	.76	1.52
June 21,	0	1.39	0 25	.69	.70	2.36
June 26,	.05	1.05	0 20	.71	.34	3.88
June 26,	.05	1.70	4 15	.99	.71	3.88
July 8,	.05	2.50	1 20	1.78	.72	3.14
July 8,	.05	3.85	7 25	2.37	1.48	3.14
July 8,	2.50	.75	0 30	.52	.23	5.64
July 9,	0	.22	1 50	.09	.13	7.33
July 9,	0	.81	1 30	.57	.24	7.32
July 14,	0	.56	2 25	.12	.44	4.58
Aug. 6,	0	1.06	0 23	.33	.73	2.52
Aug. 7,	0	.77	0 25	.31	.46	3.62
Nov. 17,	.07	.69	0 18	.25	.44	2.21
Dec. 27,	.20	2.31	6 50	1.64	.67	.60
Feb. 3, 1943	0	.60	2 30	.17	.43	.24
Mar. 16,	0	.48	3 10	.23	.25	1.08

TABLE 1 (CONT'D)

(1)	(2)	(3)	(4)		(5)	(6)	(7)
Mar. 16,	.48	.34	1	20	.24	.10	1.54
Mar. 19,	0	.52	3	30	.36	.16	1.89
May 6,	.95	1.00	2	10	.39	.61	1.93
May 8,	0	.30	1	30	.12	.18	2.80
May 10,	0	.41	1	35	.36	.05	3.61
May 15,	.35	.35	0	12	.15	.20	3.90
May 16,	0	.29	0	07	.16	.13	4.03
May 17,	0	5.05	29	00	3.96	1.09	4.30
May 17,	.50	4.55	24	00	3.55	1.00	4.75
May 17,	.50	1.85	6	40	1.44	.41	4.75
May 17,	2.65	1.00	1	45	.81	.19	6.50
May 17,	3.70	1.65	7	40	1.29	.36	7.50
May 19,	0	.60	8	15	.47	.13	8.17
June 5,	0	.87	2	45	.12	.75	4.46
June 6,	0	1.43	3	30	.82	.61	5.04
June 10,	0	.73	0	50	.34	.39	4.87
Apr. 10, 1944	.06	.53	1	05	.08	.45	1.17
Apr. 22,	.22	4.67	14	00	3.15	1.51	2.15
Apr. 22,	0	1.01	4	00	.68	.33	6.90
May 22,	0	1.06	2	10	.19	.87	2.47
May 23,	0	.59	1	10	.13	.46	2.31
May 23,	0	.41	1	40	.13	.28	2.91
Aug. 30,	0	1.70	2	00	.22	1.48	1.93
Feb. 21, 1945	.13	.85	4	45	.18	.67	.86
Mar. 1,	0	1.49	16	20	.59	.90	1.86
Mar. 6,	0	.64	4	00	.38	.26	3.92
Mar. 25,	0	1.78	18	30	.75	1.03	2.13
Mar. 29,	0	1.32	1	55	.61	.71	2.97
Mar. 30,	0	1.43	21	00	.96	.47	4.32
Apr. 1,	0	.54	6	15	.34	.20	5.14
Apr. 12,	.10	1.31	5	00	.42	.89	3.14
June 9,	0	.52	0	13	.27	.25	3.12
June 12,	0	.87	1	30	.33	.54	3.29
June 16,	0	2.20	1	50	1.33	.87	3.16
June 16,	0	.32	0	18	.18	.14	5.42
June 17,	0	.24	2	00	.14	.10	5.65
June 17,	0	.34	4	40	.20	.14	5.89
Oct. 21,	0	1.44	6	30	.22	1.22	1.09
Nov. 12,	.05	.64	0	40	.16	.48	1.01
Nov. 26,	0	.57	1	35	.12	.45	1.12
Dec. 24,	0	1.10	17	30	.33	.77	.56
Jan. 11, 1946	0	.53	4	45	.24	.29	1.32
Feb. 12,	0	1.72	21	00	.87	.85	.40
Mar. 5,	.10	1.40	5	00	.74	.66	1.11
Mar. 17,	0	1.37	14	00	.65	.72	1.29
May 3,	0	.64	0	30	.27	.37	2.24
May 17,	0	1.05	6	05	.35	.70	2.73
Aug. 5,	0	1.12	0	38	.17	.95	2.41
Aug. 14,	.15	1.49	3	40	.32	1.17	4.05
Aug. 14,	.20	3.09	2	20	2.31	.78	5.61
Aug. 14,	.20	5.00	6	05	3.41	1.59	5.61
Aug. 14,	.20	1.10	1	15	.67	.43	5.61
Aug. 15,	0	1.45	0	50	1.10	.35	9.20
Aug. 15,	.30	3.66	10	40	2.54	1.12	10.50
Oct. 31,	.35	1.49	6	30	.14	1.35	1.15
Oct. 31,	0	4.01	15	45	1.75	2.26	2.60
Nov. 2,	0	.91	4	20	.50	.41	6.37

TABLE 1 (CONT'D)

(1)	(2)	(3)	(4)		(5)	(6)	(7)
Nov. 6,	0	1.05	9	30	.44	.61	6.40
Nov. 9,	0	1.45	8	00	.89	.56	6.40
Nov. 15,	.06	.50	5	00	.15	.35	5.59
Dec. 12,	0	1.00	6	40	.70	.30	1.31
Jan. 29, 1947	0	.93	3	25	.47	.46	.32
Apr. 3,	0	.48	4	30	.25	.23	1.22
Apr. 5,	0	.53	3	30	.39	.14	1.57
Apr. 10,	0	.81	7	05	.33	.48	1.70
Apr. 24,	0	1.59	15	00	.98	.61	1.77
Apr. 29,	0	1.37	1	43	.95	.42	3.16
Mar. 1, 1948	0	.55	3	55	.10	.45	.77
Mar. 21,	0	.91	6	00	.34	.57	.96
Mar. 22,	0	1.13	11	20	.59	.54	1.78
Mar. 26,	0	.57	1	35	.30	.27	2.39
Mar. 30,	0	.70	6	50	.30	.40	2.68
Mar. 31,	0	.26	2	00	.17	.09	3.20
May 15,	0	.76	1	20	.18	.58	2.04
June 26,	0	.84	0	50	.12	.72	1.76
June 27,	0	.59	0	30	.13	.46	2.63
July 15,	0	.54	0	10	.10	.44	3.07
July 19,	0	.52	0	30	.14	.38	3.55
July 20,	0	.85	1	10	.35	.50	4.23
July 25,	0	2.45	6	10	1.10	1.35	4.59
July 25,	0	1.85	1	45	.85	1.00	4.59
Jan. 3, 1949	0	.98	1	05	.34	.64	.02
Jan. 18,	0	1.81	12	00	1.01	.80	1.34
May 9,	0	.81	0	15	.13	.68	.98
May 9,	0	1.10	0	23	.58	.52	1.78
May 21,	0	2.02	2	10	.68	1.34	2.32
June 26,	.15	1.35	1	05	.27	1.08	1.18
June 27,	.04	1.00	0	30	.41	.59	2.30
Aug. 2,	0	1.86	1	10	.49	1.37	1.47
Oct. 5,	.45	1.67	4	00	.22	1.45	1.83
Oct. 11,	1.10	.85	5	00	.22	.63	3.85
Dec. 11,	0	.42	0	20	.10	.32	2.89
Dec. 25,	0	.59	4	15	.25	.34	2.70
Jan. 3, 1950	0	4.66	25	00	3.65	1.01	2.23
Jan. 14,	0	.59	5	30	.46	.13	5.19
Jan. 25,	0	.55	2	40	.13	.42	2.89
Mar. 11,	.25	1.07	5	10	.60	.47	1.30
Mar. 12,	0	.54	5	00	.33	.21	2.33
Mar. 19,	0	.52	6	40	.32	.20	2.31
Apr. 28,	.10	.74	2	50	.19	.55	1.54
May 29,	0	.86	1	20	.24	.62	1.57
June 3,	0	1.50	6	30	.62	.88	2.35
June 3,	.50	.98	3	00	.51	.47	2.85
June 13,	0	1.27	1	30	.33	.94	2.49
June 14,	0	.62	1	40	.26	.36	3.58
Feb. 18, 1951	0	.52	3	00	.19	.33	1.72
Feb. 20,	0	1.65	15	30	.70	.95	2.12
June 16,	0	1.74	2	00	.62	1.12	2.42
June 19,	0	.85	1	45	.14	.71	3.88
June 22,	.10	.50	0	25	.11	.39	4.55
June 23,	0	.92	0	13	.21	.71	4.42
June 28,	0	.92	0	35	.43	.49	4.46

TABLE 1 (CONT'D)

(1)	(2)	(3)	(4)	(5)	(6)	(7)
July 22,	0	1.04	0 20	.18	.86	1.33
July 23,	0	.90	0 15	.38	.52	2.20
July 23,	0	.69	0 15	.42	.27	3.15
Sept. 12,	0	.90	0 35	.12	.78	1.29
Jan. 26, 1952	.10	.34	0 15	.11	.23	.39
Feb. 1,	0	.75	5 00	.27	.48	.53
Mar. 31,	.22	1.36	0 35	.97	.41	1.50
Mar. 31,	.22	2.73	2 47	2.01	.72	1.50
Mar. 31,	1.65	1.25	0 30	1.04	.21	3.15
Apr. 3,	.07	1.28	13 30	1.00	.28	3.49
Apr. 12,	0	1.55	17 00	1.30	.25	3.20
June 9,	0	2.27	1 15	.80	1.47	.30
June 9,	0	3.11	2 15	1.24	1.87	.30
June 9,	0	3.60	3 15	1.45	2.15	.30
June 9,	2.30	.78	0 15	.42	.36	2.90
June 9,	3.12	.50	0 30	.22	.28	3.42
July 2,	.15	2.92	1 10	1.11	1.81	1.62
July 2,	.75	2.34	0 30	1.11	1.23	2.22
July 2,	0	.76	0 23	.27	.49	4.54
July 16,	0	1.55	2 17	.40	1.15	3.31
Mar. 3, 1953	.07	1.05	7 00	.40	.65	1.15
Mar. 17,	0	.55	4 30	.11	.44	1.35
Mar. 30,	0	.78	5 00	.11	.67	1.05
Mar. 31,	0	.36	0 35	.12	.25	1.73
Apr. 23,	.27	1.20	1 05	.60	.60	1.49
June 26,	0	2.78	2 40	.58	2.20	.26
June 26,	0	1.58	0 45	.14	1.44	.26
June 26,	1.58	1.20	1 00	.44	.76	1.84
Oct. 5, 1954	0	.85	0 20	.16	.69	1.48
Oct. 11,	.85	.50	0 30	.11	.39	2.70
Oct. 11,	.10	1.55	5 00	.56	.99	3.25
Oct. 26,	.33	.62	0 45	.20	.42	2.73
Feb. 19, 1955	.40	.42	2 15	.22	.20	.64
Feb. 19,	0	.50	3 30	.18	.32	1.06
Feb. 26,	.05	.55	2 30	.11	.44	1.30
Apr. 23,	.05	.70	0 21	.15	.55	.97
July 17,	.05	1.55	0 48	.29	1.26	2.15

These studies show that differences in antecedent precipitation have considerable influence on retention during the summer but are of minor importance in the winter. The best correlation between antecedent precipitation index and retention was obtained for the period of October 15 to November 15, but this might change markedly if a larger number of points were available. These curves are based on the assumption that, for a particular season, the infiltration capacity is a function of antecedent soil moisture and is independent of rainfall intensity. Because these are mass retention curves, the slope of the tangent is a measure of the infiltration capacity rate.

Because it is the amount of precipitation retained rather than the total precipitation that adds to the soil moisture, a refinement was attempted using an antecedent retention index. The results showed no noticeable improvement.

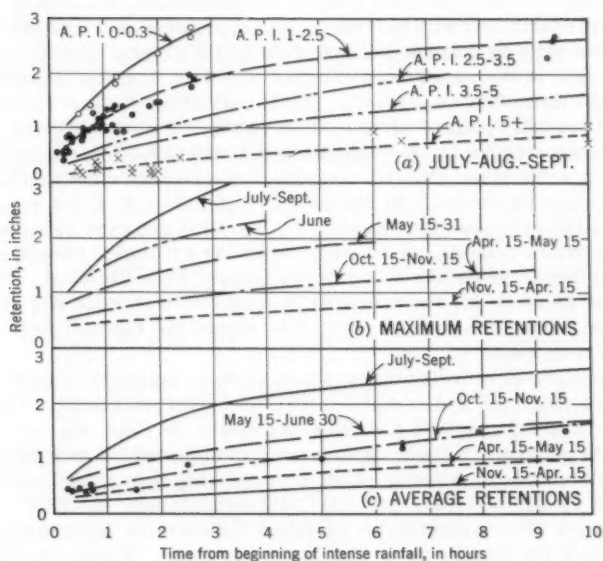


FIG. 3.—TIME-RETENTION CURVES (CLAYPEN PRAIRIES)

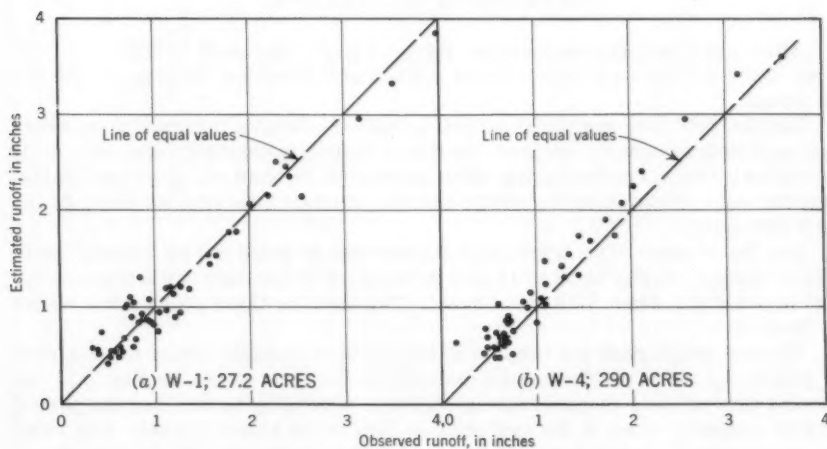


FIG. 4.—COMPARISON OF ESTIMATED AND OBSERVED RUNOFF



The investigations also indicate that data from a 187-acre watershed near Hamilton, Ohio, and the 153-acre area on the McCredie, Mo., experiment station agree quite favorably with the retention curves developed from the Edwardsville data. The soils on these areas are of the same general type as at Edwardsville, but depths to the impervious claypan may be somewhat different.

*Testing the Retention Curves.*—To test the accuracy of the retention curves, the estimated runoff volumes for a number of storms were obtained by placing the time retention curve for the appropriate season and antecedent precipitation index under the mass rainfall diagram, with the zero time and depth at the beginning of rainfall excess. In the case of watershed W-4, the mass rainfall diagram was the average of all gages. The vertical distance between the time retention and mass rainfall curves represents the estimated amount of rainfall excess, up to any time, which will eventually pass over the weir. Proper allowance must be made for the decrease in depression storage and plant interception, when, during any periods within the storm, the rainfall ceases or falls below infiltration capacity.

A large number of selected storms that gave an estimated runoff of 1/2 in. or more were tabulated and compared with the observed runoff. These comparisons for watersheds W-1 and W-4 are shown in Figs. 4(a) and 4(b), respectively. In general, the best agreement was obtained for storms occurring during the dormant season and at other times of high antecedent moisture conditions, such as July 9, 1942, May 17, 1943, and August 15, 1946. Although individual values show considerable variation between the estimated and actual runoff, the totals for all storms on W-1 agree very well. The estimated amounts on W-4 appear to be generally too high, indicating the possible need for adjustment of the time retention curves on this area.

### DEVELOPING HYDROGRAPHS

After establishing a method for estimating storm runoff volume, the next step in the analysis was to develop a method of estimating the shape of the hydrograph.

*Rainfall Intensity and the Unit Hydrograph.*—Attempts to establish an average unit hydrograph for each of the Edwardsville watersheds showed a wide variation between storms having different rainfall intensities. It was desirable, therefore, to establish relationships between rainfall intensity and shape of the unit hydrograph.

For the 27-acre area, a series of storms was selected having isolated periods of intense rainfall of 10 to 15 min duration, of reasonably uniform intensity, and producing at least 0.10 in. of runoff. The data for these storms are shown in Table 2.

The unit graph peak and time from beginning of rainfall excess to peak rate of runoff,  $t_p$ , were plotted against rainfall intensity as shown on Fig. 5. The plotted data indicate two separate groupings, depending on whether the period of high intensity came at the beginning or late in the storm period. The lines on this figure (fitted by inspection) show that both the peak of the unit graph and the time to peak are dependent on rainfall intensity and storm pattern.

To determine if similar relations existed on the larger areas, a series of storms was selected, for the 290-acre watershed, which had the following characteristics: total storm runoff not less than 0.10 in.; reasonably uniform rainfall intensity for the period of excess; and a duration of rainfall excess not



TABLE 2.—RELATION BETWEEN RAINFALL INTENSITY, UNIT HYDROGRAPH PEAK RATE AND TIME TO PEAK

Date	30-day Antecedent Precipita- tion Index	Precipitation Before Period of Excess		Precipitation Producing Unit Hydrograph			Runoff Used in Compu- ting Unit Hydro. Peak		Unit Graph Peak In/Hr	Time from Beginning of Excess Rain to Peak Rate of Runoff Minutes
		Amount Inches	Duration Minutes	Time Minutes	Amount Inches	Intensity In/Hr	Rate In/Hr	Inches		
May 27, 1938	2.93	.28	20	14	1.11	4.75	2.38	.66	3.60	12
June 10, 1938	2.77	0	0	6	.60	6.00	.61	.20	3.05	14
Aug. 11, 1939	2.64	.06	88	10	.74	4.44	.48	.22	2.18	14
April 17, 1941	2.23	.58	132	13	.42	1.95	.25	.14	1.78	20
Sept. 2,	2.55	1.02	43	12	.53	2.65	.38	.17	2.23	18
Oct. 22,	4.52	1.02	295	10	.22	1.32	.20	.10	1.40	24
June 13, 1942	1.55	.24	33	24	.92	2.30	.20	.14	1.43	33
July 8,	5.64	2.48	140	15	.60	2.40	.94	.46	2.04	19
July 9,	6.89	0	0	26	.67	1.55	.60	.63	.95	28
Aug. 6,	2.35	.03	2	18	1.02	3.40	.34	.21	1.62	27
Aug. 7,	2.49	.04	2	10	.60	3.60	.33	.21	1.57	26
June 9, 1945	2.93	0	0	8	.44	3.30	.27	.20	1.35	22
June 16,	3.11	.10	7	17	.73	2.58	.90	.60	1.50	21
June 16,	5.37	2.26	210	10	.13	.78	.12	.12	1.00	22
May 3, 1946	2.24	0	0	13	.70	3.23	.54	.32	1.69	21
Aug. 3,	1.91	.81	255	10	.66	3.96	.26	.15	1.73	24
Aug. 5,	2.41	0	0	17	.81	2.86	.26	.17	1.53	24
July 15, 1948	3.07	.09	15	10	.53	3.18	.27	.12	2.25	29
July 20,	4.44	0	0	10	.52	3.12	.53	.34	1.56	23
July 20,	5.05	.63	52	17	.27	.95	.25	.21	1.19	30
May 9, 1949	.98	0	0	10	.80	4.80	.22	.08	2.75	15
May 9,	1.80	0	0	18	1.26	4.20	1.40	.75	1.87	20
June 26,	1.18	.37	62	13	.73	3.37	.44	.22	2.00	27
June 23, 1951	4.55	0	0	12	.75	3.75	.98	.40	2.45	34
July 22,	1.29	0	0	19	1.15	3.63	.66	.40	1.65	23
July 23,	2.20	0	0	15	.95	3.80	.92	.59	1.56	23
July 31,	3.15	0	0	15	.75	3.00	1.07	.61	1.75	25
Mar. 31, 1952	1.52	.19	80	35	1.36	2.33	1.51	1.24	1.22	39
Mar. 31,	3.00	1.75	207	23	1.18	3.08	2.02	1.08	1.87	25

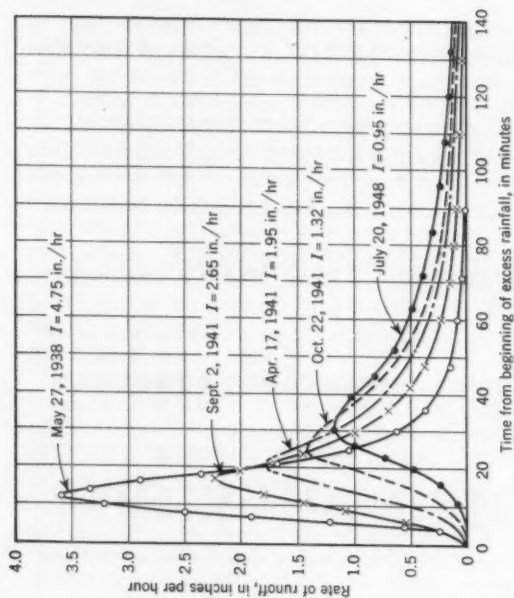


FIG. 6.—UNIT HYDROGRAPHS FOR WATERSHED W-1

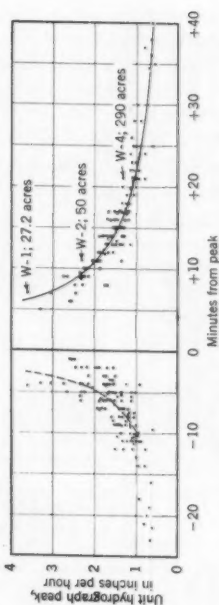


FIG. 7.—TIMES FROM PEAK OF HYDROGRAPH TO 60% OF PEAK RATE

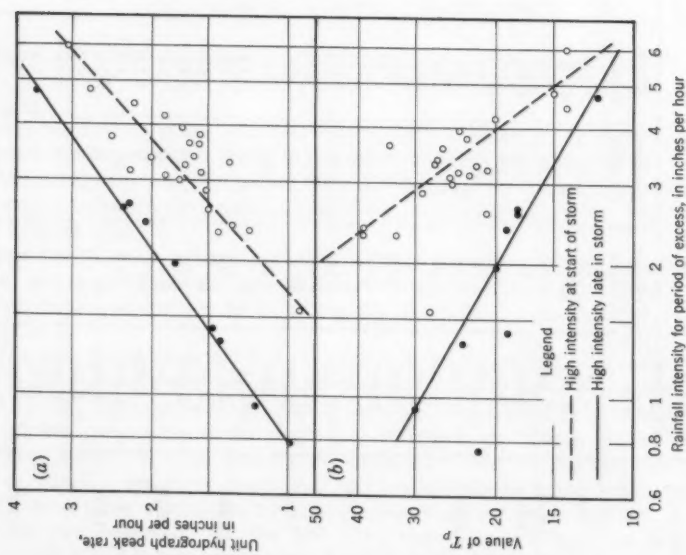


FIG. 5.—EFFECT OF RAINFALL INTENSITY AND STORM PATTERN ON THE UNIT HYDROGRAPH FOR WATERSHED W-1

less than 10 min, or more than the time to peak. The group of storms selected had a duration of rainfall excess from 11 to 30 min with the exception of one 54-min period on April 22, 1944.

Grouping of the data according to the duration of the rainfall producing the unit graph, and plotting similar to Fig. 5, indicated that duration has little bearing on the results so long as it is less than the time to peak. The plotted points again showed two separate groupings depending on whether the period of high intensity occurred early or late in the storm period. A comparison of the plottings for these two watersheds showed that, for the larger watershed, rainfall intensity has less effect on the peak of the unit graph and the time to peak. One might logically expect that the runoff hydrograph peak resulting from rainfall of a given duration would be more closely related to the net rainfall intensity, that is, rainfall intensity minus rate of infiltration. Attempts to correlate this net rainfall intensity with the peak of the unit graph and time to peak gave no improvement. This was probably due to the difficulty of making reasonable estimates of the infiltration rates for such short periods.

From the graph of Fig. 5, a series of storms was selected to represent various rainfall intensities. Unit hydrographs for each of these storms are shown in Fig. 6. The actual total runoff, for the storms selected, varied from 0.10 to 0.18 in., with the exception of the storm of May 27, 1938, which had a total runoff of 0.66 in. The original unit hydrograph definition by L. K. Sherman,<sup>3</sup> intended for use on very large drainage areas states:

"For the same drainage area, however, there is a definite total flood period corresponding to a given rainfall and all one-day rainfalls, regardless of intensity, will give the same length of base of the hydrograph."

In the data presented here, which applies to very small areas, the base of the unit graph increases as the rainfall intensity decreases. Perhaps this is because, for rains of less than 15 min., at low intensities, the soil must be at a higher moisture content to produce a minimum of 0.10 in. of runoff than for rains of higher intensities. Only a small percentage of the runoff in these storms was due to base flow or interflow since the rate 3 hr after the peak was only about 1% of the peak and, therefore, no attempt was made to separate the hydrographs into their component parts.

*Developing Synthetic Unit Hydrographs.*—With a limited amount of data available and with considerable scatter of points, as shown in Fig. 5, one may frequently find it impossible to pick a storm representative of a desired intensity or storm pattern.

A study was made of the unit graphs to show times before and after the peak when various percentages of the peak rate occurred. The data for watershed W-1 are shown in Table 3. Similar data from the other two Edwardsville watersheds were included in this analysis. The study showed two distinct groupings when the times were compared with rainfall intensity. However, all points fell in one group when plotted against the unit hydrograph peak. A plotting of the times of occurrence of rates equal to 60% of the peak is shown in Fig. 7, in which - indicates minutes before peak and + indicates minutes after peak. The results show that, when times are plotted against the unit graph peak, one curve can adequately define the shape of the recession for all three watersheds.

<sup>3</sup> "Streamflow from Rainfall by Unit-Graph Method," by L. K. Sherman, Engineering News-Record, Vol. 108, 1932, pp. 501-505.

TABLE 3.—TIMES FROM PEAK OF HYDROGRAPHS TO POINTS WHERE DISCHARGE EQUALS A GIVEN PERCENTAGE OF THE PEAK WATERSHED

Date	Rainfall		Unit Graph Peak In/Hr	Time in Minutes from Peak to--										15% Before	Peak After
	Time Min	Intens. In/Hr		80% Before	Peak After	60% Before	Peak After	40% Before	Peak After	25% Before	Peak After				
Mar. 15, 1938	26	1.20	1.36	3	8	5	15	7	27	9	43	11	67		
May 27,	14	4.75	3.60	3	5	4.3	7	5.3	10	6	15	7	20		
May 28,	12	.55	2.34	4	6	5.5	9.5		7	15	8	30	46		
June 10,	6	6.00	3.05	3	4	4	7	6	12	8	21	9	31		
Aug. 11, 1939	10	4.44	2.18	1	5	1.5	10	2	21	2	32	3	48		
Apr. 17, 1941	13	1.95	1.78	3	6	6-	12	8	20	10	33	12	47		
Sept. 2,	12	2.65	2.23	4	5	5	9	7.5	16	10	24	11	35		
Oct. 22,	15	1.68	1.89	5	7	7	11	9	18	11	27	12	38		
Oct. 22,	25	.91	1.27	8	8	12	16	15	25	18	40	21	58		
Oct. 22,	10	1.32	1.40	4	10	7-	17	9	28	12	39e	13	60		
June 13, 1942	24	2.30	1.43	6	11	9	17	13	27	17	38	22	52		
July 8,	15	2.40	2.04	3	6	4.5	11	5.5	20	7	33	9	57		
Aug. 6,	18	3.40	1.62	4	8	6	14	9	24	12	37	15	58		
Aug. 7,	10	3.60	1.57	5	9	8-	17	9	29	11	52	13	69		
June 16, 1945	17	2.58	1.50	2	8	3	13	4	21	6	39	7	60		
June 16,	12	1.50	1.19	5	8	7-	17	8	30	9	50	11	79		
May 3, 1946	13	3.23	1.69	3	4	4	8	5	15	7	32	9	53		
Aug. 3,	10	3.96	1.73	1	6	3	12	7	21	11	34	13	51		
Aug. 5,	17	2.86	1.53	4	8	6	16	8	32	11	44	14	60		
Aug. 5,	20	1.65	1.54	3	7	4	14	5	24	6	40	9	63		
June 26, 1948	20	2.40	2.35	3	5	4	9	6	15	8	30	10	40		
July 15,	10	3.18	2.25	2	4	3	10	5	20	7	30	8	41		
July 20,	10	3.12	1.56	3	7	3.5	15	4.5	26	6	42	7	65		
July 20,	17	.95	1.19	4	10	8	18	10	32	12	52	15	78		
May 9, 1949	10	4.80	2.75	2	4	4	9	6	14	7	22	8	30		
May 9,	18	4.20	1.87	4	6	6	10	7	16	8	23	10	47		
June 23, 1951	12	3.75	2.45	1	3	2	7	3	14	3	23	4	35		
July 22,	19	3.63	1.65	4	6	5	12	6	21	9	34	11	55		
July 23,	15	3.80	1.56	1.5	7	2	13	3	21	5	37	6	63		
July 23,	15	3.00	1.75	2	3	3	8	5	17	6	30	8	56		
Mar. 31, 1952	23	3.08	1.87	4	7	6	12	10	19	10	30	11	38		
Averages				3.45	6.72	5.20	12.3	7.03	21.3	8.84	33.7	10.8	50.8		
Ratio				.514		.423		.33		.264		.213			

Times before the peak of the unit graph, being affected by rainfall intensity and storm pattern, are not as consistent as times after peak, which are the result of depletion of detention volumes after cessation of rainfall. The average ratio of times before to times after the peak, for various percentages of the peak, was determined for each of the watersheds. The results, shown in Fig. 8, indicate considerable variation between watersheds. However, since less than 25% of the runoff occurs before the peak, the use of an average curve should not introduce serious errors.

The data and analysis thus far is confined to those areas on which runoff measurements are available. The results will have much wider application by adapting them to ungaged areas. Points representing the unit graph peak and time to peak for rainfall intensities of 1, 2, 3, 4, and 5 in. per hr, taken from the curve of Fig. 5, and similar plottings on other watersheds at Edwardsville and a 189-acre area near Hamilton, Ohio, are shown on Fig. 9. These curves apply to those cases where the period of high intensity occurs late in the storm.

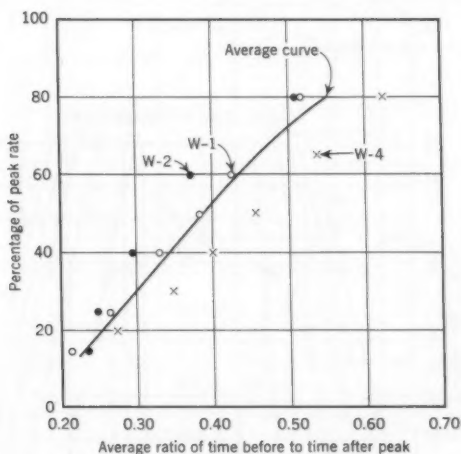


FIG. 8.—RATIO OF TIMES TO PEAK

The curves show the effect of size of area on the unit hydrograph peak and time to peak. A similar set of curves can be prepared for storms having the intense period at the beginning. The curves for times after the peak, at which various percentages of the peak rate was reached, were taken from plottings similar to those in Fig. 7. The curves showing times before the peak were determined by applying the average ratios from Fig. 8 to the times after the peak.

A series of synthetic unit hydrographs for a particular size area and different rainfall intensities can be obtained from Fig. 9.

To illustrate this, assume a unit hydrograph is required for a rainfall intensity of 4 in. per hr and a drainage area of 200 acres. To develop the synthetic unit hydrograph, enter Fig. 9 at 200 acres and go vertically to the rainfall intensity of 4 in. per hr, then horizontally from these points to find a peak rate of 1.85 in. per hr and time to peak of 22 min after the beginning of rainfall excess. Proceed horizontally along the line for a unit graph peak of 1.85 in. per hr to read times before and after the peak at the percentages shown.

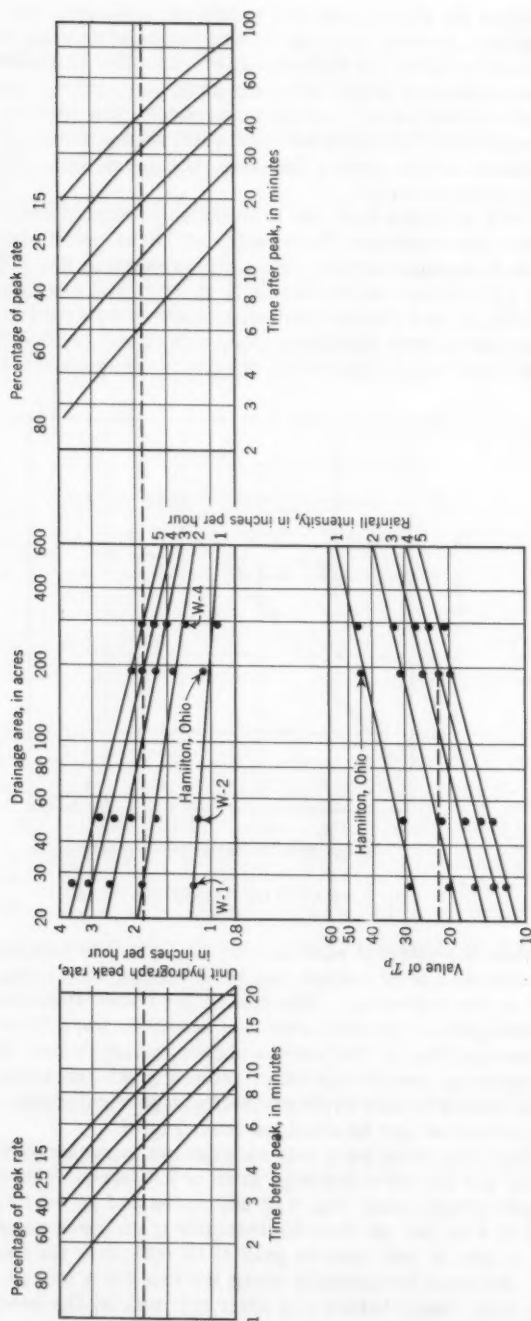


FIG. 9.—EFFECT OF RAINFALL INTENSITY AND SIZE OF WATERSHED ON THE UNIT HYDROGRAPH



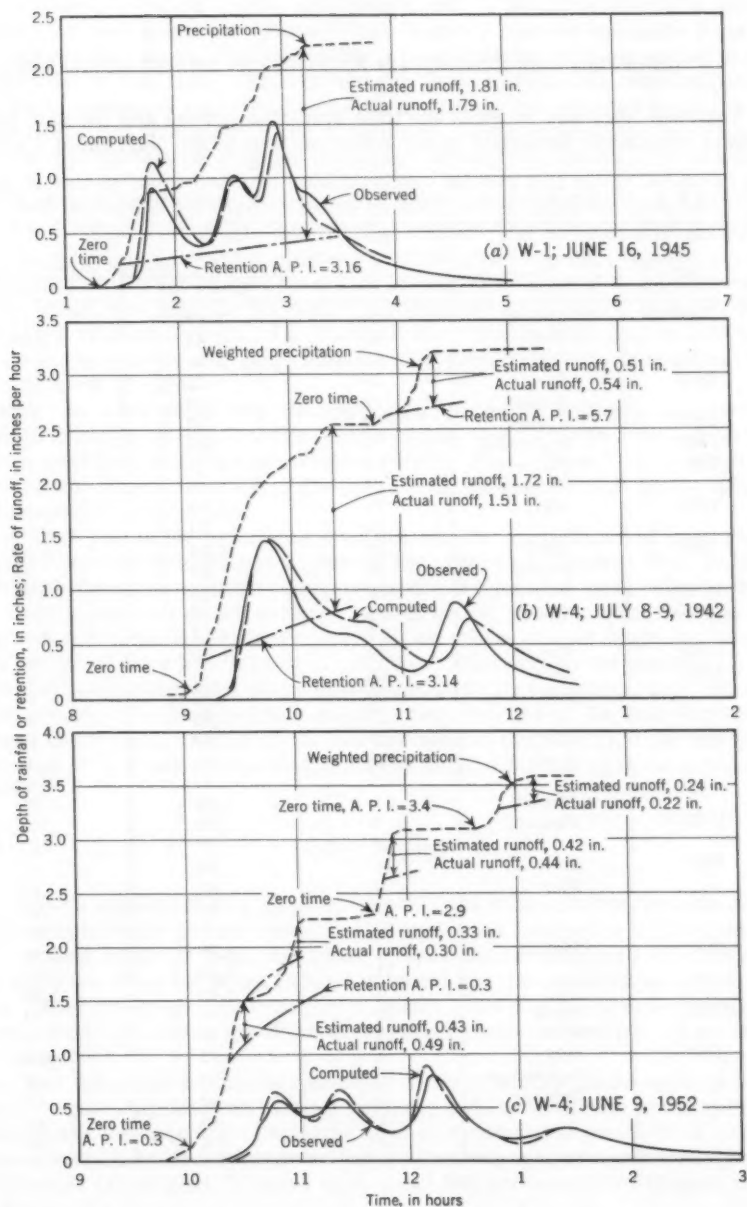


FIG. 10.—COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS

Thus a rate of 60% of the peak would occur 5 min before and 12 min after the peak.

### COMPARISON OF COMPUTED AND OBSERVED HYDROGRAPHS

The final step was to apply the time-retention curves and the unit hydrographs previously developed to the actual mass rainfall diagram of a number

TABLE 4.—COMPARISON OF OBSERVED AND COMPUTED PEAK DISCHARGES

Year	Month	Day	Peak Discharge inches/hour	
			Observed	Computed
1938	March	30	2.29	2.16
	May	27	2.41	2.64
	July	17	1.10	1.00
1939	July	16	.72	.63
	August	17	.76	.88
1940	August	4	.72	.84
1941	September	2	.48	.60
1942	July	8	1.42	1.86
	December	27	.47	.43
1943	May	17	1.19	1.06
		17	.92	1.24
1944	April	22	1.05	1.34
	August	30	.31	.30
1945	June	16	1.55	1.40
1946	August	14	2.57	2.87
		14	1.33	1.41
1947	April	29	1.50	1.60
1948	June	26	.40	.37
	July	20	.53	.50
		25	.83	.77
1949	May	9	1.39	1.56
	June	27	1.19	1.04
		28	2.07	1.83
	August	2	1.00	.92
1950	January	3	.70	.54
		3	.78	.65
1951	June	16	.84	.97
		22	.64	.92
		23	.85	.87
		23	1.01	.87
1952	March	31	1.36	1.45
		31	2.05	2.16
	July	2	1.82	1.89
1953	April	23	.51	.73
	June	26	.72	.74
1954	July	4	.92	.63
	September	20	.56	.43
	October	5	.40	.48
		11	.49	.46
		11	.83	.88

of storms on both the 27.2-acre and 290-acre areas to see how the computed hydrographs agreed with the actual record. The method used is illustrated in Fig. 10(a) for the storm of June 16, 1945 on the 27.2-acre area (W-1). The mass rainfall diagram shows considerable variation in rainfall intensity during

the excessive part of the storm. The antecedent precipitation index was computed and used to select a time-retention curve, to be plotted directly below the mass rainfall diagram. The vertical distance between the mass rainfall and the retention curve represents the potential runoff to be expected from rainfall prior to that time. The difference in these ordinates at selected points gives the increment of runoff to be applied to a unit graph selected from Fig. 6.

The first trial was made using a series of 12 min intensities even though the breaks in the rainfall diagram indicated considerable variation in intensity during certain of these periods. Investigations showed that much better agreement with the actual hydrograph could be obtained by using variable time intervals such that the intensity during any interval was more nearly uniform. The time intervals selected, as indicated on the mass rainfall diagram, varied in multiples of two and ranged from 4 to 12 min. For each of these intervals, the unit graph was selected for which the intensity most nearly matched that of the mass rainfall diagram. For example, the intensity from 2:11 to 2:33 was 1.70 in. per hr and the unit graph used for this period was the one based on the storm of April 17, 1941.

Using the same procedure but applying a series of synthetic unit hydrographs obtained from Fig. 9 to the storms of July 8-9, 1942 and June 9-10, 1952 on watershed W-4 gave the comparisons shown in Fig. 10(b) and 10(c). Note that the retention curves for each of the periods of excess is based on the API at the beginning of that period.

The final test of the principles involved was the computation of peak rates for one or more of the outstanding storms each year on watershed W-1. Table 4 shows that the difference between observed and computed peak rates is less than 10% for one-half of the storms. The greatest disagreement between the observed and computed hydrographs was for those storms beginning with a short period of very high intensity. A slight difference in the position of the time retention curve makes considerable change in the estimated runoff for this initial period. The high rainfall intensity requires use of the unit graph with a short, sharp peak, similar to the one derived for the May 27, 1938 storm on watershed W-1, which greatly magnifies any small discrepancy in the estimated runoff.

### CONCLUSIONS

1. There appears to be a good relationship, for this soil type, between antecedent precipitation and the amount of precipitation retained in a given time.
2. Runoff from the most important storms can be estimated with reasonable accuracy from the mass rainfall diagram and time-retention curves.
3. These investigations show that on small watersheds, one unit hydrograph cannot adequately define the shape of the hydrograph derived from a storm of unit duration.
4. For the small watersheds involved in these studies, there appears to be a good relationship between rainfall intensity and the peak rate of the unit hydrograph and the time from beginning of excess rainfall to peak rate of runoff.
5. A method has been presented for constructing synthetic unit hydrographs for drainage areas from 20 acres up to about 500 acres and for different rainfall intensities.
6. Computed hydrographs show close agreement with the observed record if the unit hydrographs used are based on storms having similar rainfall intensities and distribution patterns.

7. There is need for further research along these lines, particularly to see if similar relations exist for other soil types.

#### ACKNOWLEDGMENTS

These studies were conducted by the Agricultural Research Service, Watershed Technology Research Branch, L. M. Glymph, Chief, in cooperation with the Illinois Agricultural Experiment Station. A. T. Lenz, J. W. Howe, L. L. Kelly, and H. N. Holtan read the manuscript or reviewed the material and offered several helpful suggestions.

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DISCUSSION

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TWO METHODS TO COMPUTE WATER SURFACE PROFILES<sup>a</sup>

Discussion by S. V. Chitale and K. Arunachalam, F. Paderi

Closure by Joe M. Lara and Kenneth B. Schroeder

S. V. CHITALE<sup>1</sup> and K. ARUNACHALAM.<sup>2</sup>—The authors are to be congratulated for the lucid exposition they have given of the step-by-step method adopted in the Bureau of Reclamation for backwater computations when both the channel and overbank discharges are required to be considered. Two methods have been illustrated, Method B having shown to be more accurate of the two. Two assumptions have been made in the development of method; one is that  $h_f$  is the same for channel and overbank flow in a given cross section, and the other that velocity head is also the same for both channel and overbank flow in a cross section. In some cases, these assumptions may not appear to be valid and their justification would depend on the magnitude of error involved by these assumptions. Application of another step-by-step method without recourse to the above assumptions has been illustrated below. It is contended that this method is more general than Method B and its adoption in preference to Method B may be found desirable in specific cases. For comparison, an example is solved by both the methods; that is, the general method as well as Method B (Table 3). Since the data of Red Fox River given by the authors are not sufficient to illustrate the more general method advocated below, another river—Brahmani River in the State of Assam, India—has been chosen for working out comparative results.

*General Method*

**Assumption.**—The only assumption made is that water level across a cross section is same over the channel and overbank portions. This assumption is common to this as well as Method B.

**Equations.**—Besides those given by the authors, the following additional equations are required to be adopted:

Using the subscripts c and s to distinguish properties of channel and overbank portion, at any section,

$$Q_c + Q_s = Q \dots\dots\dots (1)$$

$$Q_c = K d_c \left( \frac{h_{fs}}{L_c} \right)^{\frac{1}{2}} \dots\dots\dots (2)$$

<sup>a</sup> April, 1959, by Joe M. Lara and Kenneth B. Schroeder.

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$$Q_s = K d_s \left( \frac{h_{fs}}{L_s} \right)^{\frac{1}{2}} \dots \dots \dots (3)$$

$$\Delta H_c = \Delta H_s = \Delta H \dots \dots \dots (4)$$

$$h v_{c2} - h v_{c1} = \Delta h v_c \dots \dots \dots (5)$$

$$h v_{s2} - h v_{s1} = \Delta h v_s \dots \dots \dots (6)$$

$$h_{fc} = \Delta H + \Delta h v_c - \text{eddy loss} \dots \dots \dots (7)$$

and

$$h_{fs} = \Delta H + \Delta h v_s - \text{eddy loss} \dots \dots \dots (8)$$

*Design Data.*—Backwater computations were carried out for the Brahmani River in Assam State, India:

1. A plan map in Fig. 1 of the river shows the location and designation of the different cross sections.

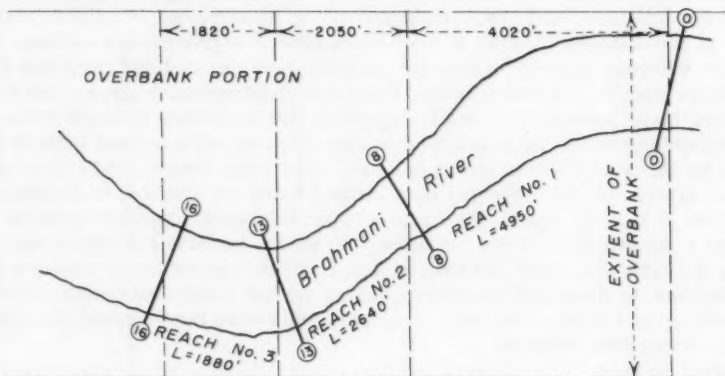


FIG. 1

2. The water surface elevation at Cross Section 0-0 is 682.00.
3. Discharge in the channel portion of cross section 0-0 at water surface El. 682 is 835,000 cfs.
4. Discharge in the overbank portion of cross section 0-0 at water surface El. 682 is 40,000 cfs.
5. "n" = 0.03 for the main channel and "n" = 0.05 for the overbank section.
6. Values of  $K_d$ ,  $A$ ,  $R$ , etc., for different water levels are calculated and given in Table 1.
7. The relationships between water surface elevations and  $k d_s$  for different cross sections are shown graphically in Fig. 2.

#### *Procedure.*

Step 1.—To start with, discharges occurring in the overbank and channel portions are known at the end section, that is, at cross section 0-0. In the first trial for estimating the discharges passing through the two portions at cross section 8-8, the velocity head is ignored, that is,  $h_f = \Delta H$ . Since  $\Delta H$  is same for both the portions,  $h_f$  overbank =  $h_f$  channel. A trial  $\Delta H$  is assumed, Col. 2, Table 2.

Step 2.—The values of  $K_d$  for the two sections 0-0 and 8-8 are averaged and the discharges  $Q_c$  and  $Q_s$  are computed by Eqs. 2 and 3 and entered in Cols. 10 and 17.

Step 3.—If the sum of  $Q_c$  and  $Q_s$  is not equal to the actual total discharge of 875,000 cfs, the actual discharge is divided in the ratio of  $Q_c$  and  $Q_s$  obtained in step 2 and adopted (Cols. 19 and 20) for finding the velocity head in the next step.

TABLE 1.—BRAHMANI RIVER

Cross Section No.	Elev.	A	P	R	K d	Reach length	h v	Bankful Stage
(a) Properties of Channel Portion								
0-0	680.00	63,408	1,413.0	44.87	39,665,756	4,950	2.8174	
	682.00	66,172	1,415.6	46.75	42,509,164		2.5880	
	684.00	68,936	1,418.0	48.61	45,479,809		2.3837	
8-8	683.30	48,230	1,167.8	41.30	28,546,541	2,640	4.8700	
	686.00	51,330	1,171.6	43.81	31,601,720		4.3000	
	688.00	53,626	1,174.6	45.65	33,944,904		3.9390	
13-13	686.00	60,307	1,265.8	47.64	39,219,254	1,980	3.1150	
	688.00	62,759	1,268.4	49.48	41,901,816		2.8760	
	690.00	65,211	1,271.4	51.29	44,604,793		2.6640	
16-16	686.20	73,700	1,638.4	44.98	46,177,066		2.0860	
	688.00	76,920	1,640.8	46.88	49,566,117		1.9150	
	690.00	80,104	1,643.2	48.77	52,990,611		1.7660	
(b) Properties of Overbank Portion								
0-0	680.00	13,060	4,366	3	815,000	4,950		R.L. 677.00
	682.00	21,800	4,370	5	1,910,000			
	684.00	30,520	4,374	7	3,330,000			
8-8	682.00	19,520	4,888	4	1,470,000	2,640		R.L. 678.00
	684.00	29,280	4,892	6	2,870,000			
	686.00	39,040	4,896	8	4,630,000			
13-13	684.00	22,460	4,502	5	1,950,000	1,980		R.L. 679.00
	686.00	31,444	4,506	7	3,430,000			
	688.00	40,428	4,510	9	5,420,000			
16-16	684.00	17,816	4,462	4	1,340,000			R.L. 680.00
	686.00	26,724	4,466	6	2,620,000			
	688.00	35,632	4,470	8	4,240,000			

Step 4.—Again a trial  $\Delta H$  is assumed. This is added to the water surface elevation at cross section 0-0 to get water surface elevation at cross section 8-8.

Step 5.—The velocity heads  $h_{v_c}$  and  $h_{v_s}$  using the above water surface elevation are calculated by using the discharges obtained in step 3 (Cols. 4 and 11).

TABLE 2.—BACKWATER COMPUTATIONS FOR BRAHMANI RIVER USING

Cross section	Trial $\Delta H$	Water surface elevation	$\frac{v^2}{2g} = h_{vc}$	$\Delta h_v$	Eddy loss	$h_{fc}$	$K_d \times 10^6$	$K_{d_{avg}} \times 10^6$	$K_d \sqrt{h_f} + \sqrt{L} = Q_c \times 10^5 \text{ cfs}$	$\frac{v^2}{2g} = h_v$	$\Delta h_v$
1	2	3	4	5	6	7	8	9	10	11	12
0-0		682.00	2.46				42.51		8.35	0.052	
8-8	1.50	683.50				1.50	29.00	35.75	6.20		
	1.50	683.50	4.50	2.04	1.02	2.52	29.00	35.75	8.05	0.065	0.013
	1.60	683.60	4.50	2.04	1.02	2.62	29.25	35.87	8.30	0.065	0.013
13-13	2.00	685.6				2.00	39.25	34.25	9.60		
	3.40	687.0	2.56	-1.94	0.19	1.27	41.00	35.13	7.75	0.087	0.022
	3.20	686.8	2.30	-2.20	0.22	0.78	40.75	35.00	6.05	0.370	0.30
	3.60	687.2	2.06	-2.44	0.24	0.92	41.20	35.23	6.60	0.30	0.24
	3.80	687.4	2.05	-2.45	0.24	1.11	41.50	35.38	7.20	0.30	0.24
16-16	1.50	688.9				1.50	50.80	46.15	12.70		
	1.50	688.9	1.65	-0.40	0.04	1.06	50.80	46.15	10.70	0.056	-0.244
	1.10	688.5	1.655	-0.395	0.04	0.665	50.20	45.85	8.40	0.06	-0.24
	1.00	688.4	1.66	-0.39	0.04	0.57	50.00	45.15	7.80	0.06	-0.24

TABLE 3.—BACKWATER COMPUTATIONS FOR BRAHMANI RIVER

Section	Reach length	Water surface elevation	Area	$K_d \times 10^6$	$\frac{K_d}{\sqrt{L}} \times 10^6$	$Q + \sum \frac{K_d}{\sqrt{L}} = h_f$
1	2	3	4	5	6	7
0-0		682.00	66,172	42.51	0.600	
			21,800	1.91	0.030	1.92
8-8	4,950	682.75	46,538	28.25	0.400	
	4,020		23,120	2.00	0.032	4.05
Trial II		684.00	49,034	29.50	0.420	
Trial III			29,280	2.90	0.064	3.27
		684.15	49,140	29.27	0.420	
			29,770	2.95	0.065	3.22
13-13	2,640	686.00	60,307	39.22	0.765	
	2,050		31,444	3.43	0.076	1.04
Trail II		687.00	61,533	41.00	0.800	
Trial III			35,932	4.30	0.095	0.95
		687.30	61,899	41.25	0.805	
Trial IV			37,278	4.55	0.100	0.935
		687.34	61,990	41.25	0.805	
			37,291	4.55	0.100	0.935
16-16	1,980	689.00	78,512	51.00	1.140	
	1,820		40,086	4.85	0.116	0.485
Trial II		688.80	78,352	50.75	1.140	
			39,186	4.70	0.110	0.490

## GENERAL METHOD SUGGESTED BY S. V. CHITALE AND K. ARUNACHALAM

Eddy loss	$h_{fs}$	$K_d \times 10^6$	$K_{d_{avg}} \times 10^6$	$K_d \sqrt{h_f} + \frac{L}{L} = Q_s \times 10^5 \text{ cfs}$	$\frac{Q}{Q_c + Q_s} \times 10^5 \text{ cfs}$	$Q_c \times 10^5 \text{ cfs}$	$Q_s \times 10^5 \text{ cfs}$	$\left(\frac{Q}{Q_c + Q_s}\right)^2 h_{fc}$
13	14	15	16	17	18	19	20	21
		1.90						
	1.50	2.50	2.20	0.40	1.32	8.20	0.55	Velocity head neglected
0.006	1.507	2.50	2.20	0.425	1.03	8.30	0.45	2.67
0.006	1.607	2.55	2.22	0.44	1.00	8.30	0.45	1
	2.00	3.10	2.82	0.875	0.82	7.9	0.85	Velocity head neglected
0.01	3.41	4.30	3.40	1.39	0.96	7.45	1.30	1.17
0.15	3.35	4.10	3.30	1.32	1.18	7.15	1.60	1.09
0.12	3.72	4.50	3.50	1.49	1.08	7.15	1.60	1.18
0.12	3.92	4.62	3.56	1.54	1	7.20	1.55	1
	1.50	4.80	4.71	1.35	0.622	7.9	0.85	Velocity head neglected
0.024	1.23	4.80	4.71	1.23	0.735	7.85	0.90	0.57
0.024	0.836	4.50	4.56	0.98	0.935	7.85	0.90	0.58
0.024	0.826	4.43	4.52	0.965	1.00	7.80	0.96	0.57

## USING METHOD B AS USED BY JOE M. LARA AND KENNETH B. SCHROEDER

$(K_d + \sqrt{L})(\sqrt{h_f}) = Q$	V	$V^2 Q \times 10^5$	hv	$h_{v1} - h_{v2}$	Eddy loss	Mean $h_f$	Total loss	H
8	9	10	11	12	13	14	15	16
8.35	12.6	1,320.00						
0.40	1.83	1.34	2.37					
8.08	17.00	2,340.00						
0.67	2.90	5.60	4.17	1.80	0.90	2.99	3.89	2.09
7.60	15.40	1,790.00						
1.15	3.93	17.70	3.22	0.85	0.43	2.59	3.02	2.17
7.57	15.35	1,790.00						
1.18	3.94	17.70	3.22	0.85	0.43	2.57	3.00	2.15
7.95	13.20	1,380.00						
0.80	2.55	5.20	2.46	-0.76	0.08	2.13	2.22	2.98
7.78	12.60	1,230.00						
0.97	2.70	7.10	2.20	-1.02	0.10	2.09	2.19	3.21
7.80	12.60	1,230.00						
0.95	2.55	7.10	2.20	-1.02	0.10	2.07	2.17	3.19
7.80	12.60	1,230.00						
0.95	2.55	7.10	2.20	-1.02	0.10	2.07	2.17	3.19
8.00	10.20	8.20						
0.75	1.87	2.62	1.52	0.68	0.07	0.71	0.78	1.46
8.00	10.40	8.65						
0.75	1.91	2.73	1.54	0.66	0.07	0.713	0.783	1.443

Step 6.—The terms  $\Delta h v_c$  and  $\Delta h v_s$  are calculated by Eqs. 5 and 6 and entered in Cols. 5 and 12.

Step 7.—Eddy loss is assumed at 50% of  $\Delta h v$  for retarded flow and 10% of  $\Delta h v$  for accelerated flow; Cols. 6 and 13.

Step 8.—The values of  $h_{fc}$  and  $h_{fs}$  are then computed by Eqs. 7 and 8 and entered in Cols. 7 and 14.

Step 9.—The value of  $K d$  for the two sections 0-0 and 8-8 are averaged and discharges  $Q_c$  and  $Q_s$  are computed.

Step 10.—The discharges obtained in Step 9 still do not satisfy Eq. 1. To get an idea of  $h_f$  to be used in the next step, the following equation is used:

$$Q^2 = K d^2 \frac{h_f}{L} \dots \dots \dots (9)$$

$$\left( \frac{Q_{\text{actual}}}{Q_{\text{computed}}} \right)^2 = \frac{h_f}{h_{f_{\text{computed}}}}, \text{ approximately } \dots \dots \dots (10)$$

Solving for  $h_f$ :

$$h_f = \left( \frac{Q_{\text{actual}}}{Q_{\text{computed}}} \right)^2 h_{f_{\text{computed}}} \dots \dots \dots (11)$$

This assumes that  $K d$  remains constant for small change in water surface elevations.

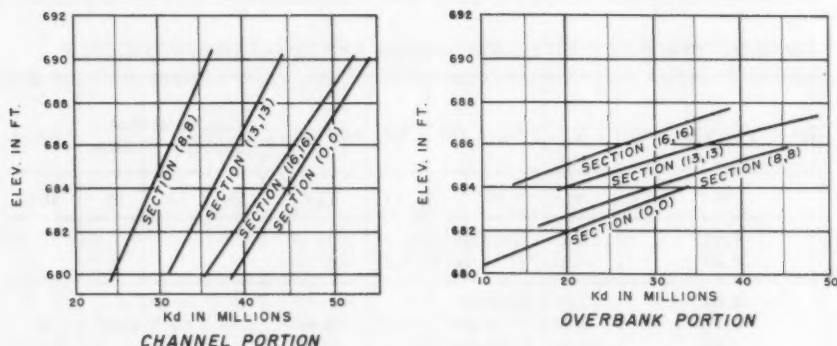


FIG. 2

As the flow in the channel usually predominates,  $h_f$  used in Eq. 11 is that for channel portion. The terms  $Q_{\text{actual}}$  and  $Q_{\text{computed}}$  are the total discharges. The value of  $h_f$  obtained by Eq. 11 is entered in Col. 21 for comparison with the assumed  $h_f$ , Col. 7 so as to give an idea of  $\Delta H$  to be used in the next step.

Step 11.—As we proceed, the discharges  $Q_s$  and  $Q_c$  approach the correct value. This procedure is repeated until Eq. 1 is satisfied. The same procedure is used to compute the upstream water surface elevations for cross sections 13-13 and 16-16.



F. PADERI.<sup>3</sup>—The writer has had occasion to compute water surface profiles for flow in open channels. He particularly appreciated the clear and accurate exposition on the two methods of finite differences derived and is pleased with them for several reasons, among them being:

1. The development of Methods A and B is based on a profound and extensive analysis of the various physical phenomena involved and the relative effects of each in the theoretical analysis.

2. The authors point out that practical judgment, in addition to theoretical analysis, also plays an important part in the evaluation of these phenomena.

3. The writer noted in this paper several ideas he developed when dealing with the same subject.<sup>4,5</sup>

The writer has nothing to add to the first two above statements, but some explanation of the third follows:

1. A fundamental element of computing by finite differences as presented by the authors is the hypothesis of linear variation of the function  $S_f$  on each subdivision, specifically its equivalent

$$\frac{Q^2}{A^2 X R}$$

where  $X$ , defined by Manning has a value in metric units:

$$X = \frac{R^{\frac{1}{6}}}{n} \dots \dots \dots (12)$$

and in American units:

$$X = \frac{1.486 R^{\frac{1}{6}}}{n} \dots \dots \dots (13)$$

Then in metric units:

$$S_f = \left( \frac{Q n}{A R^{\frac{2}{3}}} \right)^2 \dots \dots \dots (14)$$

and in American units:

$$S_f = \left( \frac{Q n}{1.486 A R^{\frac{2}{3}}} \right)^2 \dots \dots \dots (15)$$

The linear variation hypothesis, on each subdivision, of the ratio

$$\frac{Q^2}{A^2 X^2 R}$$

<sup>3</sup> Asst. Prof., Univ. of Pisa, Italy.

<sup>4</sup> "Sul moto permanente delle correnti a superficie libera," by F. Paderi, *Ricerche di Ingegneria*, Roma, marzo-aprile, 1939.

<sup>5</sup> "Sul profili di rigurgito di correnti lente," *Stab. Tip. gia Chiari, Succ. C. Mori.*, Firenze, 1946, e *Bollettino Tecnico Ingegneridi Firenze*, aprile-maggio, 1948.

(not of each element such as A or R, as used in previous studies) was already stated, to the best of the writer's knowledge, for the first time in the writer's report of July, 1946, and presented at the Commission for the Professorship (Chair of Hydraulics: D. M. June 28, 1946, and August 6, 1946).

The above-mentioned hypothesis was used in the same report (applying the method of finite differences) for the computation of water surface profiles for subcritical flow in cylindrical channels. An example was shown in Section 5 of the report applicable to specific conditions.

The writer appreciates the authors' confirmation of the validity and practicality of the forementioned hypothesis, implicit in the evaluation of  $h_f$ , for each subdivision, with

$$h_f = (L \div 2) (S_{f1} + S_{f2}) \dots \dots \dots (16)$$

2. A second element as important to practical hydraulic analysis as the one discussed previously concerns the proper introduction in the computations of a simplified and practical concept of eddy loss incident to the conversion of kinetic to static head and vice versa in open channel flow. This is usually considered as some portion of the change in velocity heads.

In previous work<sup>4,5</sup> (dealing mostly with subcritical flow) the writer developed this same concept which was discussed in general terms. Based on previous studies of this concept, the writer introduced in his conclusions another coefficient,  $\rho$  (analogous to the authors' coefficient C) combined with the coefficient  $\chi$  of uniform flow to evaluate the friction head and eddy losses holding to the true phenomena. Regarding the choice of the  $\rho$  value whether constant or variable according to the type and condition of flow, the writer pointed out evidence for the necessity of conducting further research in particular the carrying out of proper experimental tests. These tests should be geared to attain a better evaluation, both quantitatively and qualitatively, of the  $\rho$  coefficient.

The writer also explored the results of particular values of  $\rho$ , useful in practical applications ranging from zero to one and reciprocal departures in the water surface profiles. For specific conditions, interesting evaluations of the coefficient  $\rho$  or C, also designated as  $-\rho$  or  $k$ , were obtained in later studies<sup>6,7</sup> on this important subject.

Regarding these studies and those of the authors, it is important to note that minute numerical departures are not bound to a particular single set of conditions which can be said of the general agreement of the fundamental physical concepts involved. Under certain circumstances the value of  $\rho$ , C, etc., could also possibly be accounted for in the value of  $\chi$  itself which has  $n$  as the variable factor.

It is well to note also that with the introduction in Bernoulli's energy equation of the eddy losses affected by the coefficient  $\rho$ , C,  $-\rho$ , or  $k$ , there remains in the said equation a single multiplier  $1 - \rho$  (or  $1 - C$ ,  $1 + \rho$ , etc.) of the difference in velocity heads. Bernoulli's equation as quoted in paper can be written as follows considering only the eddy losses mentioned above:

$$Z_2 + d_2 = Z_1 + d_1 + h_f + (1 - C) (h_{v1} - h_{v2}) \dots \dots \dots (17)$$

<sup>6</sup> "A duzzasztási görbek számítására ajánlott módszerek" hidromechanikai összefoglalása," by Gy. Kovacs, *Vízügyi Közlemények*, N. I. Budapest, 1952, pp. 84-92.

<sup>7</sup> "Etude et tracé des écoulements permanents en canaux et rivières," by R. Silber, Dunod, Paris, 1954, p. 13.

Instead of the factor  $(1 - C)$ , a value of  $(1 + C)$  for this coefficient can be used, thus, an algebraic change in sign is noted. Based on the 100% value reported by the authors for retarded flow conditions, the factor is resolved as follows:

$$1 - \rho = 1 - C = 1 + \rho = 1 - k = 0 \dots \dots \dots (18)$$

The authors are commended on their calling attention to the numerical value of  $C$  which can change to fit the conditions observed and its assigned value is left to the judgment of the hydraulic engineer.

JOE M. LARA<sup>8</sup> and KENNETH B. SCHROEDER.<sup>9</sup>—The writers appreciate the interesting discussions submitted on the methods of computing water surface profiles.

Messrs. Golding, Anand, and Silvester introduced the idea of expressing conveyance as  $K$  equal to  $1.486 A R^{2/3}$ . Adjustment of the  $K_d$ -values used by the writers can be readily facilitated in cases involving variations in " $n$ ." This is particularly adaptable to conditions where the water surface profiles have been observed for a series of discharges which results in a definite " $n$ "-stage relationship. The writers found this idea very helpful in subsequent studies.

Mr. Golding's additional suggestion of replacing Col. 5 of Table 2 with two columns is also useful when applying the preceding analysis. He suggests specifically tabulating the values of  $K = 1.486 A R^{2/3}$  in one column and  $K_d = K/n$  in the other. Mr. Golding's graph plotting the conveyance " $k$ " vs. depth indeed would be helpful in the case of an infinitely wide flood plain. The graphs prepared by Mr. Silvester in nomographical form, wherein the solution of the equation,  $Q = (K/n) (S^{1/2})$  is included for various prismatic channels also would be helpful in certain cases.

The refinement of using different flow travel paths was questioned by Messrs. Golding and Peterson of Method B. This method has not been tried in sufficient cases by the writers to arrive at a more conclusive appraisal. In the few cases tried, it was also found by the writers that computed elevations were not significantly affected. The method merits though from the standpoint of considering the existent hydraulic conditions realistically, yet conforming to standard theoretical analysis.

The writers acknowledge with interest the work carried out by Mr. Golding in the study wherein the " $n$ " values were increased based on observed data. He concluded that when the deflection angle exceeded 45 degrees, the " $n$ " value justifiably could be increased. Although Mr. Anand referred to the formula, bend loss = coeff.  $\times 1/2 (h_{v1} + h_{v2})$ , he did not offer any suggestions or guides to assigning a numerical value for the coefficient. This, as with other hydraulic assumptions, undoubtedly is left to the judgment of the engineer.

Messrs. Anand and Peterson offer further discussion of the eddy loss analysis. From actual hydraulic measurements, Mr. Anand verified his selection of eddy loss coefficients which probably supports his recommending that eddy losses be neglected for converging reaches but a 50% correction used for a diverging reach. Mr. Peterson, on the other hand, suggests a 10% correction for converging reaches and from zero to 100% for diverging reaches.

Mr. Peterson's presentation on developing the coefficient  $a_1$ , and his analysis of the continuity equation resulting in the formula,  $Q = K_d g S_g^{1/2}$ , are of academic interest. His example of computing the water surface profiles is

<sup>8</sup> Hydr. Engr., Bur. of Reclamation, Denver, Colo.

<sup>9</sup> Hydr. Engr., Bur. of Reclamation, Denver, Colo.

another approach incorporating the application of this  $\alpha$  correction and use of the geometric mean of the  $K_d$ s. The results, however, are quite different from those of the writers. About a 50% error in discharges is indicated.

It is to be remembered that in the final analysis of any method, regardless of the mathematics involved in satisfying Bernoulli's energy equation, the computed results carry only the same degree of accuracy as that of the observed field data. The moment eddy losses and corrections for velocity head changes are introduced in any of these computations, the element of judgment by the individual engineer plays a significant role. The writers believe, therefore, that greater emphasis be placed upon gathering field data which are judged representative of the hydraulic conditions prevalent in the reach under investigation. Whatever mathematical analysis is applied to these data is at the discretion of the engineer. Undoubtedly many of us have our "pet" methods. The following comments apply to the discussion submitted by Messrs. Chitale and Arunachalam:

It was not the writers' intention to show Method B was more accurate than Method A. The primary reason for presenting the two methods was to show computational procedures covering situations where the flow lengths between sections were different in the main channel and overbank area. However, as mentioned previously, the results, applying Method A and using only the flow length of the main channel, do not vary significantly from those using Method B as applied to various problems thus far.

There are several comments regarding the supporting data that require explanation or clarification as listed below:

a. There appears to be an error in the reach lengths quoted in Table 1. They are listed as being identical for the main channel and overbank portion. Reach lengths shown in Fig. 1 do not seem representative of the flow path taken by the water in the overbank areas. It seems more reasonable that the flow would follow more of sinuous path somewhat parallel to the main channel yet shorter.

b. It would be interesting to know how the hydraulic properties of each cross section (Figs. 1 and 2, Table 1) for the overbank portion were determined. The important question is: What happens when the bankfull stage is reached and then surpassed? Normally, the cross section is laid out in the field in a straight line extending to the limit of the overbank portion. If this were done in the example, Sections 13-13 and 16-16 would intersect in the overbank area imposing some complexity in the computation of the hydraulic characteristics.

c. A lesson in significant digits is to be gained in the tabulation of the various hydraulic characteristics listed in Table 1. In general, these can be held to three digits yet be compatible and representative of the gathered field data. A look at the tabulated data shows that they range from one place in R of the overbank portion to eight places in the  $K_d$  values for the main channel. There is some inconsistency.

d. Velocity head ( $h_v$ ) values appear in Table 1 for all elevations of the main channel of the four cross sections. No mention is made as to how or why these values were tabulated. A check shows they were all computed on a basis of a discharge equal to about 855,000 cfs.

e. In Table 1 there is also listed values for a term R.L. at each section. It is presumed this means "reference level." However, to list them as bankfull stage is another unexplainable item.

With exception to some of the above comments, Messrs. Chitale's and Arunachalam's method is judged meritorious and appears to be a worthwhile procedure. The results of the discussers' and writers' procedures are plotted in Fig. 3 to show comparative differences.

Mr. Paderi presents an interesting discussion of the results he attained in his studies involving finite differences. The writers have not had the opportunity to review any of the literature quoted by Mr. Paderi. It is noted, however, that most of his experiments and studies dealt with subcritical flow which

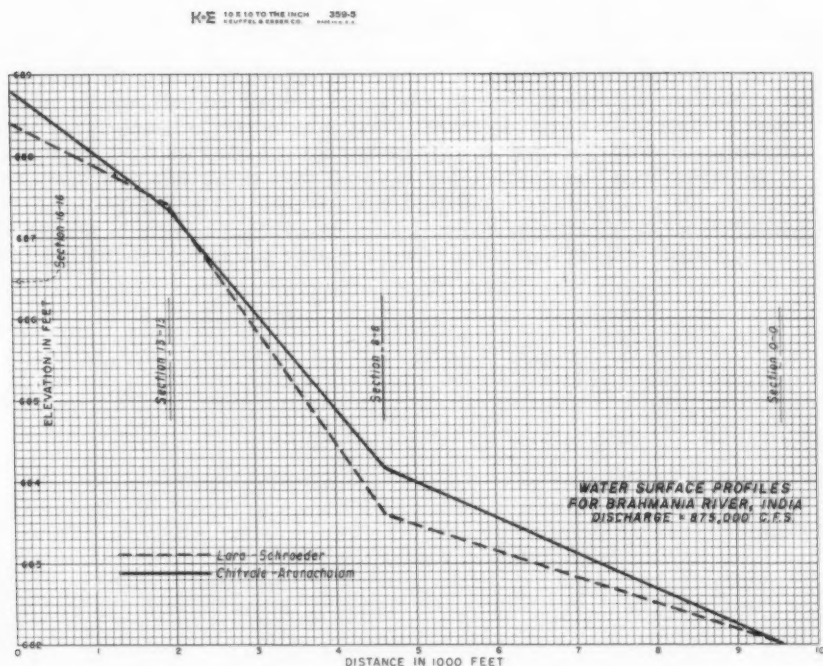


FIG. 3

the writers believe he meant flows in the "shooting" range or at supercritical velocities. Mr. Paderi's comments relative to the coefficient assigned for making eddy loss corrections are covered by previous discussors. The writers appreciate Mr. Paderi's remarks on the opinion he shares regarding the fact that there is still a lot of responsibility riding on the engineer's shoulders in the selection of these coefficients and it might be added that this applies also to the assignment of "n" values for use in Manning's formula.

The first of these is the fact that the majority of the population of the United States is now living in urban areas. This is a result of the process of urbanization, which has been going on since the beginning of the nineteenth century.

The second of these is the fact that the majority of the population of the United States is now living in the South and West. This is a result of the process of migration, which has been going on since the beginning of the nineteenth century.

The third of these is the fact that the majority of the population of the United States is now living in the middle class. This is a result of the process of social mobility, which has been going on since the beginning of the nineteenth century.

The fourth of these is the fact that the majority of the population of the United States is now living in the industrial sector. This is a result of the process of industrialization, which has been going on since the beginning of the nineteenth century.

The fifth of these is the fact that the majority of the population of the United States is now living in the service sector. This is a result of the process of deindustrialization, which has been going on since the beginning of the twentieth century.

The sixth of these is the fact that the majority of the population of the United States is now living in the post-industrial sector. This is a result of the process of post-industrialization, which has been going on since the beginning of the twentieth century.

The seventh of these is the fact that the majority of the population of the United States is now living in the information sector. This is a result of the process of informationization, which has been going on since the beginning of the twentieth century.

The eighth of these is the fact that the majority of the population of the United States is now living in the knowledge sector. This is a result of the process of knowledgeization, which has been going on since the beginning of the twentieth century.

The ninth of these is the fact that the majority of the population of the United States is now living in the creative sector. This is a result of the process of creativeization, which has been going on since the beginning of the twentieth century.

The tenth of these is the fact that the majority of the population of the United States is now living in the service sector. This is a result of the process of serviceization, which has been going on since the beginning of the twentieth century.

The eleventh of these is the fact that the majority of the population of the United States is now living in the post-industrial sector. This is a result of the process of post-industrialization, which has been going on since the beginning of the twentieth century.

The twelfth of these is the fact that the majority of the population of the United States is now living in the information sector. This is a result of the process of informationization, which has been going on since the beginning of the twentieth century.

The thirteenth of these is the fact that the majority of the population of the United States is now living in the knowledge sector. This is a result of the process of knowledgeization, which has been going on since the beginning of the twentieth century.

The fourteenth of these is the fact that the majority of the population of the United States is now living in the creative sector. This is a result of the process of creativeization, which has been going on since the beginning of the twentieth century.



## FLOODS OF THE FLORIDA EVERGLADES<sup>a</sup>

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Closure by Edwin W. Eden, Jr.

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EDWIN W. EDEN, JR.<sup>1</sup>—Any discussion of the Everglades is incomplete without information on the Seminoles—its natural inhabitants—and their role in the development of this unique area. There can be no doubt that the Seminoles have learned to live in the extremely wet, hot, and humid environment. The flood problems arose when development by the “white man” required relief from the extremely frequent and long-duration flooding. Mr. Burnet’s discussion is greatly appreciated as it provides background on the Seminoles in an extremely interesting phase—the development of the Everglades.

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<sup>a</sup> June, 1959, by Edwin W. Eden, Jr.

<sup>1</sup> Chf., Planning and Reports Branch, Engrg. Div., U.S. Army Engrg. Dist., Jacksonville Corps of Engrs., Jacksonville, Fla.

the use of Eq. 8 for the solution of  $K$ . Since this is an analytical expression, the value of  $K$  can be solved to any desired accuracy. However, Eq. 8 is a rather involved mathematical expression, a solution for  $K$  is not an easy matter. It is believed that the graphical methods will provide a much easier and faster solution for those who are not mathematically inclined. For the sake of greatest accuracy in using the graphical method, a probability paper of large scale should be used.

The study made by Mr. Sammons on the accuracy of the graphical methods is very interesting. When either the original variate or the log-transformed variate is used, however, the result should be theoretically the same.<sup>2</sup> Any difference between the two approaches should resort to the personal error involved in the graphical construction.

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<sup>2</sup> Closure to "The Probability Law and Its Engineering Applications," by Ven Te Chow, Proceedings, ASCE, Vol. 82, 1956, pp. 6-7.

# DETERMINATION OF HYDROLOGIC FREQUENCY FACTOR<sup>a</sup>

Closure by Ven Te Chow

VEN TE CHOW,<sup>1</sup> M. ASCE.—The writer appreciates the constructive and interesting discussions by Messrs. Weiss and Sammons. Both gentlemen suggest the methods using the transformed variate. Mr. Weiss suggests an analytical approach, while Mr. Sammons suggests a graphical approach.

It is believed that each method has its own merits and it should best apply to a case as required by the circumstances under consideration. The analytical method requires only the direct solution of a mathematical equation. It can, therefore, produce results to any desired accuracy, but the solution of the implicit equation for the frequency factor is not an easy matter. The graphical methods replace the mathematical solution of an equation by the graphical construction. Their accuracy depends, of course, on the scale of the probability paper used in the graphical construction. The use of an ordinary probability paper should be satisfactory for practical purposes. For a theoretical study which requires high degree of accuracy, the analytical method is recommended.

When  $y$  is normally distributed, its mean  $\bar{y}$  is equal to its median  $M_y$  or

$$\bar{y} = M_y \dots \dots \dots (1)$$

which is the value of  $y$  at a probability of "50%-of-time." Since  $M$  is the median of  $x$ , it follows

$$M_y = \ln M \dots \dots \dots (2)$$

From Eqs. 1 and 2,

$$\bar{y} = \ln M \dots \dots \dots (3)$$

or

$$M = \exp(\bar{y}) \dots \dots \dots (4)$$

Substituting this expression for  $M$  in Eq. 6 in the original paper and solving for  $\bar{x}$ ,

$$\bar{x} = \exp(\bar{y} + \sigma_y^2/2) \dots \dots \dots (5)$$

Mr. Weiss uses this equation to derive an expression for a reduced variable  $t$ . The latter is essentially a frequency factor in terms of the variable  $y$ . From Mr. Weiss' Eqs. 3 and 4, the following may be written:

$$y/\bar{y} = 1 + C_v' K_y \dots \dots \dots (6)$$

where

$$C_v' = y/\bar{y} \dots \dots \dots (7)$$

and

$$K_y = t = \sigma_y/2 + (1/\sigma_y) \ln(1 + C_v K) \dots \dots \dots (8)$$

Comparing with Eq. 1 in the paper, it may be noted that Eq. 6 is an application of Chow's general formula to the transformed variable  $y$ . Mr. Weiss suggests

<sup>a</sup> July, 1959, by Ven Te Chow.

<sup>1</sup> Prof. of Hydr. Engrg. in-charge, Univ. of Illinois, Urbana, Ill.



GENERALIZED DISTRIBUTION NETWORK HEAD LOSS CHARACTERISTICS<sup>a</sup>

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Discussion by G. C. Anderson, J. V. Radziul, and P. Celenza

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G. C. ANDERSON,<sup>16</sup> M. ASCE.—The author has recognized that the method of proportional loads is not applicable for determining the ability of a distribution system to supply fire-flow demands. After an arterial system, capable of satisfactorily supplying the expected domestic and industrial consumption for the design period, has been determined, it will be necessary to consider individually all possible fire-flow demands on the system. This should be done considering consumption at the maximum daily rate (the average rate of consumption on the maximum day) for the design period. The fire flow will have to be available from the arterial system at sufficient pressure so that it may be delivered through the secondary feeders and the minor distributors to the desired fire location with a residual pressure of 20 psi.

J. V. RADZIUL,<sup>17</sup> and P. CELENZA.<sup>18</sup>—The author is to be congratulated for his clear, concise documentation and presentation of a new method of calculation for the determination of overall system head losses through a wide range of demand and equalizing storage rates utilizing a minimum of network analyses. His step-by-step solution of actual sample problems should be of assistance not only to the hydraulic engineer in the design office but to the water works operator as well.

It should be noted that some water distribution network designers may not consider that the functional relations of Eq. 1 represent an entirely new concept,<sup>19</sup> since various types of approximation techniques somewhat similar in nature have been used for years, in network analyses, to obtain crude estimates of source to point system head losses under varying conditions of system input without complete network calculations. However, this paper for the first time has proven that the functional relations of Eq. 1 can be satisfied only under proportional loading. Eq. 2 is a new and entirely original contribution to the published literature. Application of the principles embodied in Eqs. 1 and 2 will improve water distribution network design and water system operation.

The writers believe that these procedures, in addition to augmenting design computation techniques, provide a simple practical tool which can be readily applied by most water works operators or engineers in appraising the limitations of an existing water distribution system from either an engineering or operational viewpoint. The author's contribution is indeed timely and valuable

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<sup>a</sup> January, 1960, by M. B. McPherson.

<sup>16</sup> Supervisory Engr., Natl. Bd. of Fire Underwriters, New York, N. Y.

<sup>17</sup> Distribution Design Engr., Philadelphia Water Dept., Philadelphia, Pa.

<sup>18</sup> Asst. Distribution Design Engr., Philadelphia Water Dept., Phila., Pa.

<sup>19</sup> "Flow and Loss of Head in Distribution Systems," by J. J. Doland, *Journal, A. W. W. A.*, Vol. 35, No. 234, March, 1943.

when one considers that practically all of the approximately 18,000 water distribution systems in these United States are experimenting "growing pains" or are in the throes of substantial rehabilitation (Of the 18,000, about 300 are major systems.).

With these procedures as an aid, the evaluation of an existing water distribution system can be accomplished without considerable expenditure of time or money for equipment. Good water works practice incorporates a yearly or bi-yearly inspection of fire hydrants. Inspection procedure could easily be amended to include a zero-flow pressure reading and the time of recording. This information, together with the elevation of the fire hydrant and normally available pumping station and storage operating records, will establish sufficient data for the use of the author's equations. Computations can then be made to determine head losses at a given point for any range of system demand and system input rates.

Operators are confronted with the urgent need for pressure data, particularly for the peak hour of maximum day consumption. This field data is difficult to obtain due to timing, coincidence of peak consumption loads and maximum operation and construction work loads, personnel and equipment limitations, and the impossibility of saturating any sizeable area with pressure gages. The need is now obviated, since head losses for any given point can be calculated directly for any system demand rate.

Hydraulic grade contour maps for different levels of consumption can be made for an existing system without recourse to a computer study or elaborate hand calculations. The hydraulic grade contours superposed on a pipe system diagram would provide a graphic picture of the system, indicating the strength or weakness of the various areas. The study undoubtedly would confirm certain known low pressure locations and would probably reveal other conditions which are not readily apparent. Comparison of the slopes of hydraulic gradients and pipe sizes would indicate those supply lines in which corresponding flows are much less than potential capacities. Overloaded supply lines would be indicated in the same manner. Those locations for which the data is obviously inconsistent with calculated values obtained via Eqs. 1 or 2 should bear further investigation in the field. The writers' experience indicates that inconsistencies can usually be traced to the influence of adverse local field abnormalities, such as closed or partially closed valves, open boundary control valves between pressure districts, or local non-proportional loads (severe fluctuation of a sizeable industrial or commercial drawoff).

In the author's discussion on proportional load characteristics without equalizing storage, he states with reference to Fig. 4 that "correlation is good except for the "Southwest" station. Part of the scatter of field data, but the probability of inexact proportionality of local loads must be considered at least partially responsible." The writers performed the network analyzer design runs for the Belmont Gravity District shown in Fig. 4 and are in full agreement with these statements. The "southwest" main is the principal feeder serving a large industrial water consumer located approximately 5.9 airline miles from the filtered water basin. During the period of field testing represented in Fig. 4, pump operation at this industrial plant caused pressure fluctuations of 3 to 10 psi; the resulting water hammer when the pump was started or stopped produced a local short-time pressure fluctuation of up to 38 psi with damped effects transmitted clear back to the filtered water basin. This situation was remedied by having the pump draw directly from an existing storage reservoir at the plant, filling the reservoir from the system. The writers are



confident that closer correlation would not be obtained with new field measurements at the "Southwest" point since "calculated" head losses to the plant (fed by 16-in. main shown at bottom of Fig. 3) are in good agreement with current operational data telemetered to the Load Control Center, which includes a continuous record of pressure at the plant.

Fig. 6A shows the proposed Belmont High Service District for the near future and incorporates proposed improvements consisting of a 20-in. reinforcing main and an elevated tank. The existing 1960 district is a closed pumping system without storage. Current data for this district covering a wide range of flow data selected at random from the log sheets of the City of Philadelphia's Load Control Center appear in Table 5. Elevation of the station discharge

TABLE 5.—BELMONT HIGH SERVICE DISTRICT (1960)

Item No.	Date-Time	P <sub>B</sub> , Discharge Pressure in psi	Q <sub>d</sub> = Q <sub>p</sub> , in mgd	Gage Pressure at Point X, in psi	$\Sigma h_1$ , in feet, from P <sub>B</sub> to Point X		
					Measured (Field Data)	Calculated	
						m = 1.85	m = 2.00
1.	3/14/60 0000	53.8	6.4	64.5	17.0	17.3	16.8
2.	3/14/60 1830	54.0	10.0	54.6	40.3	39.6	40.9
3.	3/14/60 2330	53.6	8.0	63.0	26.2 <sup>a</sup>	26.2	26.2
4.	3/20/60 1630	58.4	8.5	64.0	28.8	29.3	29.6
5.	4/2/60 0000	53.5	6.2	66.0	12.8	16.3	15.7
6.	4/2/60 0200	48.4	5.0	62.3	9.6	10.9	10.2
7.	4/2/60 1300	54.0	9.2	57.2	34.3	33.9	34.7
8.	4/3/60 0200	51.9	4.1	66.6	7.8	7.6	6.9
9.	4/3/60 2330	55.2	7.2	63.4	22.7	21.6	21.2

<sup>a</sup> Used to determine  $K_1$  for "calculated" head losses. For  $m = 1.85$ ,  $K_1 = 0.559$ . For  $m = 2.00$ ,  $K_1 = 0.409$ .

pressure gage at P<sub>B</sub> is 258.3 ft. The pressure gage at Point X is located on an 8-in. main, very near the site of the future elevated storage tank shown in Fig. 6A. This 8-in. main comprises part of the distribution grid of a small local area supported by the 12-in. main shown in Fig. 6A and an old 8-in. and an old 12-in. feeder in the vicinity of the future 20-in. main. Elevation of the field pressure gage at Point X is 216.6 ft. Item No. 3 in Table 5 was arbitrarily selected to determine  $K_1$ . Since  $Q_d = Q_p$  at P<sub>B</sub>, Eq. 1 applies. For  $\Sigma h_1 = 26.2$  ft, with  $m$  taken as 1.85,  $K_1 = 0.559$ ; with  $m$  taken as 2.00,  $K_1 = 0.409$ . The "calculated" head losses from P<sub>B</sub> to Point X, when using either  $m = 1.85$  or 2.00, compare quite favorably with the "measured." With the exception of

Item No. 5 in Table 5 "calculated" values are within 1/2 lb of the "measured" values or indeed well within the normal combined probably error of the gages. The Belmont High Service District is predominately a residential system with relatively minor industrial usage. The close agreement of the results with actual values, for this system at least, lends further validity to the use of proportional loading as a design assumption. Inconsistencies between an earlier field survey and calculations made at that time by the author led to the location

TABLE 6.—TORRESDALE HIGH SERVICE AND FOX CHASE BOOSTER DISTRICT (1960)

Item No.	Date-Time	P <sub>T</sub> , Dis-charge Pres-sure in psi	Gage Pres-sure at Point Y, in psi	P <sub>T</sub> , in mgd	Oak Lane Sta-tion, mgd	Stand-pipes A, mgd	$\frac{Q_{PT}}{Q_d}$	$\Sigma h$ , in feet, from P <sub>T</sub> to Point Y		
								Mea-sured (Field Data)		
									m = 1.85	m = 2.00
1.	3/18/60 1030	152	75.6	9.6	7.0	2.3 out	0.508	22.4	21.4	26.8
2.	3/18/60 1630	158	80.0	9.2	2.4	0.7 in	0.844	26.1	28.9	32.5
3.	3/19/60 1530	159	80.4	9.3	5.6	2.6 in	0.756	27.5	27.2	31.1
4.	3/19/60 2200	163	80.0	8.9	3.6	4.4 in	1.10	37.6	33.3	34.8
5.	4/2/60 1700	147	75.4	6.4	5.8	1.8 out	0.457	11.3 <sup>a</sup>	11.3	11.3
6.	4/3/60 0300	156	82.3	6.0	1.9	2.6 in	1.13	16.2 <sup>a</sup>	16.2	16.2
7.	6/13/60 1200	149	73.0	9.8	6.9	3.7 out	0.480	21.5	21.4	26.6
8.	6/13/60 1800	147	72.0	9.2	7.4	0.9 out	0.526	19.2	20.1	25.1
9.	6/14/60 0600	150	78.5	6.1	4.2	0.6 in	0.629	11.1	10.8	12.1
10.	6/14/60 2000	149	75.0	8.8	6.5	3.6 out	0.466	16.9	17.0	21.4

<sup>a</sup> Used to determine  $\phi$  and  $n$  for "calculated" head losses. For  $m = 1.85$ ,  $\phi = 0.542$  and  $n = 2.60$ . For  $m = 2.00$ ,  $\phi = 0.421$  and  $n = 2.55$ .

of an 8-in. and two 6-in. boundary valves which had been inadvertently opened, causing a leakage of about 15% of the High Service output to the Belmont Gravity District.

Fig. 5 shows the future Torresdale High Service District with proposed piping improvements indicated by dashed lines. The existing (1960) district is also served by storage from Standpipes at A. It incorporates an additional area, called the Fox Chase Booster District, as indicated on the lower edge of Fig. 5. This present composite service district is supplied not only from P<sub>T</sub> but also from either the Oak Lane Pumping Station or the Fox Chase Booster Station via the Fox Chase District (both pumping stations are located below the

system shown in Fig. 5). Random data for the existing district, with the Fox Chase Booster shut down, taken from the log sheets of the Load Control Center appear in Table 6. Elevation of the station discharge gage at  $P_T$  is 0 ft. The pressure gage at Point Y is located on a 12-in. grid main in the vicinity of Point B in Fig. 5. Elevation of the field pressure gage at Point Y is 154 ft. The values from Items 5 and 6 were substituted in Eq. 2. Simultaneous solution gave  $\phi = 0.542$  and  $n = 2.60$  for Point Y, for  $m = 1.85$ ;  $\phi = 0.421$  and  $n = 2.55$  for  $m = 2.00$ . Comparison of the "calculated" versus the "measured" head losses from  $P_T$  to Point Y when  $m = 1.85$  is considered good and within acceptable limits of accuracy. Moreover, the  $P_T$  discharge pressure was only obtainable to the nearest point. Load Control Center personnel stated that the error in measurement from  $P_T$  to Point Y could be as much as 1.5 psi, depending on operational circumstances. Torresdale High Service District is relatively residential in character with large wooded tracts and farm areas interspersed with localized concentrations of industry or institutional developments.

TABLE 7.—HAZEN-WILLIAMS C - VALUES - EXISTING DISTRICTS

Belmont High Service (Existing Arterial Piping in Fig. 6A)			
Diameter, inches	C	Diameter, inches	C
24	127	20	136
24	97	20	136
24	97	20	50
20	125	12	65
20	126	12	60
20	126	12	42
Torresdale High Service (Existing Arterial Piping in Fig. 5a)			
Diameter, inches	C	Diameter, inches	C
24	146	16	117
24	138	16	117
24	135	16	106
24	132	16	70
24	128	16	70
24	104	16	48
24	79	12	118
20	115	12	83
20	114	12	77
20	110	12	67
16	137	12	67

<sup>a</sup> In Fox Chase District: 20 in., C = 124; 16 in., C = 48; 16 in., C = 44; 12 in., C = 47.

Here, in this district, utilizing equalizing storage, the limitations of proportional loading are apparently applicable as well as the assumption that the pipes on the average fit  $m = 1.85$ .

The use of the author's generalized network head loss characteristics in conjunction with field data has been demonstrated in this discussion. The equations under the limitations of assumed proportional loading and the application of  $m = 1.85$  for "new" and "old" pipes regardless of their condition give "calculated" head loss values in fairly good agreement to actual "measured" values

for the two Philadelphia districts cited. In the case of the Belmont High Service District, use of  $m = 2.00$  gives "calculated" values that compare as favorably with the "measured" values as those obtained when using  $m = 1.85$ ;  $m = 1.9$  would provide the best fit between actual and calculated values. It should be noted that the gage at Point X is at the extreme end of the Belmont High Service District and is located in a weak, local distribution grid which would be subject to more severe fluctuation than a point nearer the arterial piping. For the Torresdale High Service District it was found that  $m = 1.85$  provided the best fit as opposed to any other value for  $m$ . In the writers' opinion the use of  $m = 1.85$  for network analyses is reasonable and consistent when normal design assumptions and overall accuracy of data and results is considered. In Table 7 are listed the Hazen-Williams C-values for the Belmont High Service and Torresdale High Service Districts. With such wide ranges of C-values, it appears that  $m = 1.85$  for "new" and "old" pipes is thoroughly justified.<sup>20</sup>

For flow to storage the author states that  $Q_p/Q_d > 1$ . Obviously this always holds true only when a district with equalizing storage has only one pumping station as illustrated in Fig. 5 and by Table 1. In districts with equalizing storage and multiple pumping stations  $Q_p/Q_d > 1$  only when the storage inflow rate is greater than the combined output of the other pumping stations (see Items 4 and 6 in Table 6 and the No. 4 series of runs in Table 4).

The foregoing suggests the use of the author's concepts for operational purposes. High and low "normal" operating pressure values could be determined for critical areas to obtain criteria for activating warning devices. Control centers could establish valid operating standards which would result in more efficient and economical operation. In complex systems using two or more sources of input plus equalizing storage, more effective utilization of the stations to maintain adequate district pressures would be assured regardless of tank inflow or outflow rates. Conversely, improper station operation or irregularities in the distribution system would also be indicated.

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<sup>20</sup> Discussion of Walter L. Moore's "Relationships Between Pipe Resistance Formulas," Proceedings, ASCE, March, 1959, by M. B. McPherson, Proceedings, ASCE, September, 1959, pp. 143-155.

# THE FOURTH ROOT $n$ - $f$ DIAGRAM<sup>a</sup>

Discussion by Donald Van Sickle

DONALD VAN SICKLE,<sup>23</sup> A.M. ASCE.—There has been presented a "general resistance diagram" for use with the Darcy-Weisbach formula (and with the Manning formula as well), presumably as a substitute for the much-reproduced, but apparently seldom-used, Moody diagram. The merits and demerits of the "general resistance diagram," as opposed to the common empirical formulae, have been discussed frequently in the various technical journals.<sup>24,25,26</sup>

Since Mr. Blench is presenting the diagram as a practical aid and has purposely avoided detailed theoretical explanations, further discussion of the theoretical points is beyond the scope of the paper. There are, however, a number of features of the diagram which merit some discussion.

First, the proposed diagram incorporates the Blasius equation to represent "smooth-turbulent" flow conditions, rather than the familiar von Kármán-Prandtl logarithmic relationship of the Moody diagram. Basis for this change is the belief that the logarithmic formula lacks a "completely logical foundation and verification in the range of practical data." The writer feels, however, that most of the published data appear to substantiate the von Kármán-Prandtl curve. The data presented by Peter Anton Lamont,<sup>27,28</sup> with one exception, give considerable support to the von Kármán-Prandtl curve, and the experiments on new aluminum pipe,<sup>29</sup> on a large, concrete-lined tunnel,<sup>30</sup> and on coated steel penstocks,<sup>31</sup> offer further evidence that the Blasius equation is not applicable "in the range of practical data," except at Reynolds numbers

<sup>a</sup> January, 1960, by T. Blench.

<sup>23</sup> Hydr. Engr., Turner and Collie Cons. Engrs., Inc., Houston, Tex.

<sup>24</sup> "Flow in Rough Conduits," by Henry M. Morris, Jr., Transactions, ASCE, Vol. 120, 1955, p. 373.

<sup>25</sup> "Design Methods for Flow in Rough Conduits," Proceedings, ASCE, Vol. 85, July, 1959, p. 43.

<sup>26</sup> "Relationships Between Pipe Resistance Formulas," by Walter L. Moore, Proceedings, ASCE, Vol. 85, March, 1959, p. 25.

<sup>27</sup> "A Review of Pipe-Friction Data and Formulae, With a Proposed Set of Exponential Formulae Based on the Theory of Roughness," by Peter Anton Lamont, Proceedings, of the Inst. of Civ. Engrs., Part III, Vol. 3, April, 1954, p. 248.

<sup>28</sup> "Formulae for Pipe-Line Calculations—Technical Committee Report B," Proceedings, of the Third Congress of the Internal. Water Supply Assn., London, 1955.

<sup>29</sup> "The Prediction of Flow Rates in Aluminum Pipe, Irrigation Tubing, and Fittings, and its Hydraulic Efficiency After Years of Service," by J. S. Campbell and A. Brebner, Transactions, of the Engrg. Inst. of Canada, Vol. 3, No. 1, April, 1959, p. 43.

<sup>30</sup> "Head-Loss Coefficients for Niagara Water Supply Tunnels," by J. B. Bryce and R. A. Walker, The Engineering Journal, Engrg. Inst. of Canada, Vol. 42, No. 7, July, 1959, p. 68.

<sup>31</sup> "High Velocity Tests in a Penstock," by Maxwell F. Burke, Transactions ASCE, Vol. 120, 1955, p. 863.

below  $2 \times 10^5$ . Above this limit, use of the Blasius equation will give  $f$ -values and friction losses lower than those which have been determined by experiment.

Second, Mr. Blench defines his "roughness height" in terms of a base which is different from the base for  $k$  in the Moody diagram. Most of the published roughness-height values have been expressed in terms of the "equivalent sand grain diameter" of Nikuradse. It would seem as though any departure from this established practice would cause considerable confusion and can be justified only under extraordinary circumstances. The reason given, however, is to "give  $e$ -values that suggest something like the over-all height of irregularities in unlined rock tunnels . . . in deference to . . . the preference . . . for values that suggest visual roughness heights."

This attempt to give some physical significance to the  $e$ -value is, of course, commendable. Unfortunately, the practical value of such a move is questionable since, as the author notes, the irregularities in unlined rock tunnels are "the only practical roughnesses that are fairly common and are really assessable by eye." For lined tunnels and all types of pipe conduits, then, the proposed  $e$ -values will have no more physical significance than those on which the Moody diagram is based.

It should be noted that the author "would be content to treat  $e$  as a code number to be attached to roughness types that would be recognized intuitively." Such a code number has, apparently, not prevented the Hazen-Williams and Manning equations from gaining widespread acceptance. Perhaps Mr. Blench would be willing to reconsider this change in the definition of  $e$  to prevent confusion.

Third, the proposed diagram eliminates the family of transition curves between the "smooth turbulent" and "rough turbulent" flow regions which is a characteristic feature of the Moody diagram, and replaces them with a group of curves that represent the trends for different materials. The reasons given for this change are that the family of curves represents extrapolation well beyond the range of Colebrook's data, and that the transition curves are "totally unlike ones that can be found for other materials." A comparison of Figs. 1 and 2 reveals that Colebrook's data covered a very considerable portion of the transition region. Of the twenty transition curves shown in Fig. 1, only the upper two and the lower three seem to represent a significant extrapolation. Since there is a paucity of data in both of these areas, it is difficult to attack the validity of these extrapolations, particularly in view of the abundance of data corroborating the curves in the middle portion of the transition region.

Presumably, the curves mentioned for "other materials" are those from Nikuradse's experiments with uniform sand roughness,<sup>32</sup> and from the paper by G. H. Hickox, A. J. Peterka, and R. A. Elder<sup>33</sup> with discussion, on large steel and concrete lined tunnels. As noted, these curves are unlike the Moody diagram transition curves.

It is implied that the Colebrook transition curves should be used only for "commercial iron pipes," and the other curves used for concrete pipe and large tunnels. Lamont's data,<sup>27,28</sup> however, show asbestos-cement pipe, cement-lined pipe, and concrete pipe conforming to Colebrook's transition curves. In addition, there does not appear to be any unanimity of opinion in the literature that concrete surfaces are comparable to the Nikuradse uniform

<sup>32</sup> Engineering Hydraulics, by Hunter Rouse, John Wiley and Sons, Inc., New York, 1950, p. 112.

<sup>33</sup> "Friction Coefficients in a Large Tunnel," by G. H. Hickox, A. J. Peterka, and R. A. Elder, Transactions, ASCE, Vol. 113, 1948, p. 1027.



sand surfaces. To the writer's knowledge, there are no reliable data available for concrete surfaces which indicate that the Nikuradse transition curves are truly representative of these surfaces, and he must conclude that the Nikuradse curves have been included simply to illustrate that some types of surface cannot be represented by the Colebrook transitions.

It is pointed out that the Hickox, Peterka, and Elder curves (and presumably the Ontario tunnel curve at the bottom of Fig. 2) "show the kind of trend to be expected from the N curves, but to a different degree." Hickox, Peterka, and Elder in their discussion of these data conclude, however, that "although the Apalachia tests were carefully made, they do not conform to either the Nikuradse or the Colebrook type of transition." Cyril Frank Colebrook,<sup>34</sup> in addition, in his discussion of the Ontario tunnel tests, comments that "despite an appreciable experimental scatter, the test results are in very satisfactory agreement with" the von Kármán-Prandtl smooth-pipe law.

The apparently conflicting interpretations of the meaning of these curves lead the writer to conclude that it would be less advisable to extrapolate from them to establish a curve for a particular conduit than to extrapolate from the Colebrook transition curves, for which a fairly large body of data is available. Certainly, before one could reasonably extrapolate from these new transition curves, it would need to be shown that they are truly representative of the available data on large conduits and tunnels.

An examination of the twenty-nine curves shown in Fig. 19 of Hickox, Peterka, and Elder's paper (not all of which are for large diameter conduits) indicates that about half of the large conduit curves have the same trend as the Colebrook transition curves, several are similar to the Nikuradse transition curve, and the six selected by Mr. Blench represent the most extreme departure from the Colebrook transition curve. It would seem to be difficult to describe them as being "representative."

Colebrook has also presented additional data for lined, unlined, and partly-lined rock tunnels.<sup>35</sup> In the discussion of this paper<sup>36</sup> are presented additional curves for large diameter conduits. The general trend of the curves, with the exception of the Apalachia results, follows the Colebrook transition curves, although Colebrook cautions that "the experimental scatter is generally too great . . . to determine the actual shape of the transition curve for concrete surfaces." The data for the new Niagara tunnel<sup>30</sup> fall in a pattern somewhat similar to the curve for the Ontario tunnel shown in the author's diagram. J. B. Bryce and R. A. Walker, however, make no attempt to relate these data to either the Colebrook or Nikuradse transitions. It would appear appropriate for Mr. Blench to include a number of these additional transition curves to make it clear that the data for tunnels and large conduits are not all as different from the Colebrook transition as his initial selection would suggest.

Finally, the proposed transition zone curves are said to be "widely spaced because the practical user is expected to sketch transitions in terms of (a) the given ones, (b) all other available information relevant to the specific problem, and (c) personal judgment." The wide spacing and completely different shapes

<sup>34</sup> "Turbulent Flow in Pipes with Particular Reference to the Transition Region Between the Smooth and Rough Pipe Laws," by Cyril Frank Colebrook, *Journal, Inst. of Civ. Engrs.*, Vol. 11, No. 4, February, 1939, p. 133.

<sup>35</sup> "The Flow of Water in Unlined, Lined, and Partly Lined Rock Tunnels," *Proceedings*, of the Inst. of Civ. Engrs., Vol. 11, September, 1958, p. 103.

<sup>36</sup> Discussion of "The Flow of Water in Unlined, Lined, and Partly Lined Rock Tunnels," *Proceedings*, of the Inst. of Civ. Engrs., Vol. 12, April, 1959, p. 523.

of the curves, together with the usual lack of specific roughness information available in designing a new conduit suggest that "personal judgment" would be a critical factor in the use of Fig. 2. Since most designers have not had Mr. Blench's very considerable experience in conduit design, this importance of the judgment factor would seem to limit the usefulness of the diagram. The use of additional representative transition curves would increase the diagram's value to less-experienced designers.

Of the objections to the use of the Moody diagram listed in Appendix 1, the writer questions the validity of items i, ii, and iv, but must agree with items iii, v, and vi. In connection with item iii, however, Peter Ackers<sup>37,38</sup> has published a set of design charts which considerably simplify the use of the Moody diagram. It should be pointed out, however, that the charts are based on the Colebrook transition curve and would not be applicable for different shapes of transition curves.

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37 "Resistance of Fluids Flowing in Channels and Pipes," by Peter Ackers, Hydr. Research Paper No. 1, Her Majesty's Stationery Office, London, 1958.

38 "Charts for the Hydraulic Design of Channels and Pipes," Hydr. Research Paper No. 2, Her Majesty's Stationery Office, London, 1958.

HYDROLOGIC STUDIES BY ELECTRONIC COMPUTERS IN TVA<sup>a</sup>

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Discussion by J. L. Kovner

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J. L. KOVNER.<sup>6</sup>—The use of automatic electronic computers in processing and analyzing hydrologic data is increasing at a rapid rate. Mr. Snyder has shown one cost comparison between hand and machine methods which illustrates why this development is taking place. In addition, it is pointed out that desk calculators have definite limitations in handling multiple regressions with ten independent variables.

The author mentions other basic computations such as mean daily discharge and sediment concentrations. These are not complicated calculations but the volume of data favors electronic computers. The USGS has a new water-level recorder which punches head measurement at regular time intervals on paper tape and translates them to magnetic tape for immediate automatic processing.

The general application of the machines in solving engineering and statistical problems makes it possible for TVA to have its own installation. Those not so fortunate can purchase machine time at normal rates from computer laboratories. No one engaged in extensive hydrologic investigations need forego the computing improvements described by Mr. Snyder.

The writer spent some time on the four watersheds prior to 1955, in the early stages of the experiment. The design represented quite a departure from the standard practice of using control watersheds. It was apparent, then, that the four watersheds would have to be calibrated to account for varying meteorological conditions since the 4 by 4 Latin square design would not effectively control these variables by the process of randomizing the treatments in time. The writer was skeptical then that this could be done even though it appeared that the small watersheds with uniform cover should behave almost like runoff plots. Mr. Snyder admits that the old techniques did not work and it may be said that the high speed computing machines are the *modus operandi* in the current state of affairs.

The author is certainly to be congratulated in developing this method of analysis. Although the final relationship for storm peaks is expressed explicitly in Eq. 5, one should not overlook the manner in which various hydrologic processes are taken into account, using a complicated bookkeeping system. This represents definite progress in hydrologic analysis and further application of the techniques is warranted.

It appears, however, that the paper has some shortcomings in evaluating the results. The experiment was not set up to test whether the cultural treatments would have any effect. There was enough evidence on hand to verify that in west North Carolina row crops on steep slopes would produce much higher

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<sup>a</sup> February, 1960, by Willard M. Snyder.

<sup>6</sup> Statistician, Rocky Mountain Forest and Range Experiment Sta., Forest Service, U. S. Dept. of Agric., Fort Collins, Colo.

storm peaks than pasture. The experiment could not demonstrate against these facts even on the basis of low precision. The analysis must supply information on the difference between treatments within fairly narrow confidence limits.

In this respect, the results in Table 2 are not as good as indicated, namely, that differences between peaks for various covers appear to be real; this is not enough information. With standard errors of 50% and higher—even for pasture cover—it will require great improvement in the regressions to obtain satisfactory confidence limits. For watershed No. 1, pasture, the 95% confidence limits are approximately -0.07 to 0.93 cfs. The predicted peaks used depart considerably from the means in Table 1, and this accounts for the large standard errors. These predicted values are most interesting, however, since from a land-use point of view, it is the large storm peaks that are important.

In this connection, the use of the regressions for tests of significance and confidence intervals as suggested is open to some question. The assumption of normality of the dependent variable for a net of fixed values of the independent variables would have to be verified. Storm peak data are usually significantly skewed from normal. It also appears from Table 1 that variances increase with size of peak.

It is suggested, under the heading "Equation for Peak Discharge," that for watershed 5 special analysis will be needed to adjust for changes in cover. It is evident that the regressions for corn and wheat on any watershed will have larger variances than pasture because of changes in cover from bare soil at planting time to partial plant cover at full establishment.

In effect, separate regressions computed for the storms in different months would have different parameters in Eq. 5, and a combined regression for all storms may not be justified. The analysis could probably be improved by comparing regressions on a seasonal or monthly basis.

Mr. Snyder uses an arbitrary and fixed time breakdown of the storm period. There are four periods for storm peak hydrograph analysis and five periods for storm runoff. These were evidently adopted on the basis of an overall analysis of runoff-producing storms. It would be interesting to know whether further subdivision of the storm duration into contributing periods would improve the relationships. The ultimate in electronic computing for this type of analysis appears to be a continuous flow of input data of the hydrologic and meteorological variables.

SCOUR AT BRIDGE CROSSINGS<sup>a</sup>

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Discussion by Joseph N. Bradley

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JOSEPH N. BRADLEY,<sup>13</sup> M. ASCE.—The model results on scour at bridge abutments and piers have been treated in a logical and commendable manner. The consistency with which the model results plot could be misleading however, by making the prediction of scour appear as a simple routine procedure. Since Mr. Laursen has said little with regard to the limitations of his results, a few remarks on this phase of the subject may be appropriate.

Figs. 5 and 6 represent the results of model studies on abutment scour made under essentially ideal conditions, for example, with bed of granular material free to move, a rectangular flow cross section, and a uniform velocity distribution. One can compare the author's results with those of a completely independent set of experiments made with different size and gradation of bed material, a constant depth of flow and a uniform velocity distribution, performed at Colorado State University,<sup>8</sup> and find that the two are in close agreement. This indicates that the model results are consistent and easy to duplicate.

Where streams meet similar specifications in nature as those of the models, there should be a reasonable correspondence between model and prototype. There are streams in India and Pakistan which do approach these so-called ideal conditions; the stream beds present an unlimited depth of alluvial material, the river channels are extremely wide with more or less constant width-to-depth ratio, and the velocities are low due to an unusually flat gradient (the average is 2 ft in 4 miles in East Pakistan). Under such conditions the velocity distribution cannot vary greatly across the stream. Field measurements of scour at bridge abutments, spurs, and guide banks for the rivers of India and Pakistan are on record<sup>14</sup> and these show surprisingly good correlation with the model results.<sup>15</sup>

Limited experience with scour on rivers in the United States show less favorable comparisons. The reason is obvious; the gradients are steeper, the cross sections are irregular, the velocities are higher, and the velocity distributions are far from uniform. Under these conditions the greatest scour does not necessarily occur at the abutments but is more likely to be found in

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<sup>a</sup> February, 1960, by Emmett M. Laursen.

<sup>13</sup> Hydr. Engr., Internatl. Engrg. Co., Inc., Dacca, East Pakistan.

<sup>8</sup> "Backwater Effects of Bridge Piers and Abutments," by H. K. Liu, J. N. Bradley, and E. O. Plate, Civ. Engrg. Sect. Proj. Report CER., 57HKL2, Colorado A. and M. College, Fort Collins, Colo., 1957.

<sup>14</sup> "The Behavior and Control of Rivers and Canals," by Sir Claude Inglis, Research Publication 13, Part II, Central Water Power Irrigation and Navigation Report, Poona Research Sta., 1949.

<sup>15</sup> "Field Verification of Model Tests on Flow Through Highway Bridges and Culverts," by Carl F. Izzard and Joseph N. Bradley, Proceedings of the Seventh Hydr. Conf., Iowa, 1958, pp. 225-243.

the portion of the channel where the depth of flow and velocity are greatest. Records of the United States Geological Survey of bridge sites in Mississippi, where the beds are generally of alluvial material, show this to be true. There is also evidence from past floods in various sections of the United States that the settlement of piers in the deeper portion of the channel is more common than abutment failures due to scour. Yet if Figs. 3 and 6 are consulted, in order, it is found that prediction from the model gives scour depth up to three times the pier width for the center of the channel, while scour up to six times the depth of flow is supposedly possible at abutments. The latter can result in fantastic figures, which are true in the case of the model with rectangular cross section, where all scour is concentrated at the abutments; but such predictions are unreasonable when applied to irregular cross sections in the field.

This discussion was not written to confuse the issue or discredit the model results (which are valid for the conditions tested). Rather it is to point out some of the remaining unknowns and (1) encourage investigators to make a concerted effort to take soundings of streams at constrictions both before and during floods for the purpose of better understanding the field problem and making better utilization of the model results; and (2) to warn engineers to not use the model information blindly but to treat each river crossing as an individual problem, using the model results as a guide rather than a definite solution. Returning to item (1), further model studies will not produce the answer desired; field measurements are the only alternative. A further comment on item (2) is that model results applied to abutment scour in the United States will certainly fall on the side of safety. The extra cost of unduly deep footings may, therefore, be sufficient to finance a comprehensive field study in a very short time.



TRAP EFFICIENCY OF RESERVOIRS, DEBRIS BASINS, AND DEBRIS DAMS<sup>a</sup>

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Discussion by Herman G. Heinemann

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HERMAN G. HEINEMANN,<sup>17</sup> F. ASCE.—This paper is a very helpful summary of the better-known articles on the subject of trap efficiency. It also helps call attention to the need for more study, research, and analysis on this extremely important matter.

Speaking generally, there are three sedimentation considerations of importance that face the planner or designer of a proposed reservoir: (1) How much sediment will be delivered to the proposed reservoir (sediment yield)? (2) What percentage of the delivered sediment will be retained in the reservoir (trap efficiency)? (3) Where will the retained sediment be deposited (sediment distribution)? Help on any of these questions is a definite contribution to the knowledge of reservoir sedimentation.

It is assumed that the definition of trap efficiency given in the "Introduction" was not intended. Trap efficiency is generally expressed as the ratio between sediment accumulation and sediment inflow.

For clarity purposes, it should also be pointed out that the term "sediment production" as it is used under the heading "Summary of Paper by Brune" and also in the referenced article by Glymph, has been largely replaced in the more recent years by "sediment yield." This also pertains to the heading of Col. 7 in Table 1. This is made apparent by the use of the term "sediment yield" in the later portions of this paper where reference is not made to articles or papers of a number of years ago. These terms are still used somewhat interchangeably, but sediment production is now considered as referring to erosion or sediment volume from small areas—plots—single cropped, etc., whereas sediment yield refers to the sediment volume from more complex watersheds or subwatersheds—as defined by Glymph.<sup>18</sup>

Since the trap efficiency for a structure is a changing value, high at first and lower as the reservoir becomes filled with sediment, it is the writer's opinion that a date or a period of time should always accompany the trap-efficiency value given for any reservoir; it was expressed in this way for a number of reservoirs included in the paper. One should not give a trap efficiency value for "the life of a structure" without defining this or giving the period of years considered as the useful or design life. Trap-efficiency values will be more meaningful if expressed as follows:

- (1) "The 50 yr. trap efficiency of x reservoir is estimated at Y%."
- (2) "The trap efficiency of x reservoir is Y% (1959)."

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<sup>a</sup> February, 1960, by Charles M. Moore, Walter J. Wood and Graham W. Renfro.

<sup>17</sup> Hydr. Engr., Sedimentation Studies, Watershed Tech. Research Branch, Soil and Water Conservation Research Div., Agric. Research Service, Lincoln, Nebr.

<sup>18</sup> "Studies of sediment yields from watersheds," by Louis M. Glymph, Jr., Internatl. Union of Geodesy and Geophysics, Tenth Genl. Assembly, Rome, Italy, 1954, Vol. 1, pp. 178-191, 1955, illus.

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TOLKMITT'S BACKWATER AND DRAWDOWN CURVE TABLES<sup>a</sup>

Discussion by Ven Te Chow

VEN TE CHOW,<sup>19</sup> F. ASCE.—Tolkmitt developed his tables on the assumption of a parabolic channel having the width sufficiently large compared with the depth  $y$ , so that the wetted perimeter  $P$  can be considered approximately equal to the top width  $T$ . Accordingly,  $T$  varies with  $y^{0.5}$ ; the water area  $A$  varies with  $y^{1.5}$ ; and the hydraulic depth  $D$  and hydraulic radius  $R$  both vary with  $y$ . Thus, for Tolkmitt's special case, the hydraulic exponents<sup>20</sup> can be shown as  $N = M = 4$ .

For channel sections of a general shape, the flow-profile equation<sup>20</sup> is

$$x = \frac{y_n}{S_0} \left[ u - \int_0^u \frac{du}{1 - u^N} + \left( \frac{y_c}{y_n} \right)^N \int_0^u \frac{u^{N-M}}{1 - u^N} du \right] + \text{constant} \quad (8)$$

where, using author's notation,  $y_n = d$ ,  $S_0 = s$ ,  $u = (d + z)/d$  and  $(d + h)/d$  for a raising of water level, and  $u = (d - z)/d$  and  $(d - h)/d$  for a lowering of water level. When Chezy's formula is used, it can be shown that  $y_c/y_n = sC^2/g^{1/4}$ . Thus, Eq. 8 can be written partially in author's notation as

$$x = \frac{d}{s} \left[ u - \int_0^u \frac{du}{1 - u^N} + \left( \frac{sC^2}{g} \right)^{N/4} \int_0^u \frac{u^{N-M}}{1 - u^N} du \right] + \text{constant} \quad (9)$$

where  $\int_0^u \frac{du}{1 - u^N} = F(u, N)$ , a varied-flow function. For Tolkmitt's special case, Eq. 9 is reduced to

$$x = \frac{d}{s} \left[ u - \left( 1 - \frac{sC^2}{g} \right) \int_0^u \frac{du}{1 - u^4} \right] + \text{constant} \quad (10)$$

where

$$\int_0^u \frac{du}{1 - u^4} = F(u, 4) = \frac{1}{8} \ln \left( \frac{u+1}{u-1} \right)^2 + \frac{1}{2} \tan^{-1} u \quad (11)$$

It may be noted that  $F(u, 4) = \pi/4$  when  $u$  becomes infinity.

For a raising of water level,  $X = u = (d + z)/d = 1 + z/d$  and  $u > 1$ . Thus, Eq. 11 becomes

$$F(u, 4) = \frac{1}{4} \ln \left( 1 + 2 \frac{d}{z} \right) + \frac{1}{2} \tan^{-1} \left( 1 + \frac{z}{d} \right) \quad (12)$$

<sup>a</sup> May, 1960, by R. D. Goodrich.

<sup>19</sup> Prof. of Hydr. Engrg. in charge, Univ. of Illinois, Urbana, Ill.

<sup>20</sup> "Open-Channel Hydraulics," by Ven Te Chow, McGraw-Hill Book Co., Inc., New York, 1959.

From Eq. 2

$$f(X) = u - F(u, 4) + \frac{\pi}{4} \dots \dots \dots (13)$$

In the varied-flow function table,<sup>20</sup> for  $u > 1$ , a constant,  $\pi/4$ , was subtracted from  $F(u, 4)$  to make the function value near to zero when the variable  $u$  becomes very large. Let  $F'(u, 4)$  represent the table value. Then,

$$f(X) = u - F(u, 4) \dots \dots \dots (14)$$

This relation indicates that the Tolkmitt's table can be replaced by the general varied-flow function table for flow-profile-computation purposes. For example, with  $u = 1.060$ , the table value<sup>20</sup> for  $F(u, 4)$  for  $N = 4.0$  is 0.506. By Eq. 14,  $f(X) = 1.060 - 0.506 = 0.554$ . The above relation also applies to  $X = u = (d + h)/d = 1 + h/d$ .

Similarly, the varied-flow function for a lowering of water level, with  $X = u = (d - z)/d = 1 - z/d$  and  $u < 1$ , can be shown as

$$F(u, 4) = \frac{1}{4} \ln \left( 2 \frac{d}{z} - 1 \right) + \frac{1}{2} \tan^{-1} \left( 1 - \frac{z}{d} \right) \dots \dots \dots (15)$$

From Eq. 4,

$$f(X) = F(u, 4) \dots \dots \dots (16)$$

Thus, Tolkmitt's table value is identical with the value given by the varied-flow function table. This also applied to  $X = u = (d - h)/d = 1 - h/d$ .

Tolkmitt's table is widely known in European literature. The author should be commended for introducing this table to the American literature.

HOOD INLET FOR CLOSED CONDUIT SPILLWAYS<sup>a</sup>

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Discussion by C. D. Smith

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C. D. SMITH,<sup>10</sup> F. ASCE.—It is heartily agreed that the hydraulics of culverts is not as simple as was once believed. In this regard, Mr. Blaisdell's paper represents a further useful contribution to this interesting and important subject.

One feature of the hood inlet which was not mentioned, and which should receive attention, is a discussion of the limitations of the design. In the writer's opinion, there are very definite structural and hydraulic limitations which should be considered by the designer when he is making his selection of inlet design.

The first point concerns the design of the splitter-type anti-vortex wall. In 1957, a design using the hood inlet was carried out under the direction of the writer for a 60-in. corrugated iron pipe. At that time, the only data reported for the anti-vortex device was for the splitter-type wall. According to the minimum dimensions, the splitter wall would have to cantilever upstream 13.75 ft beyond the invert of the pipe. It was found that several ring stiffeners would be required for the pipe, and structural steel angles would be required to reinforce the thin splitter wall. The design was complicated, and there was still some question as to the adequacy of the splitter wall to resist impact due to floating ice cakes or debris. Finally the design was abandoned in favor of a more conventional reinforced concrete inlet with a rounded lip.

It may be that problems similar to this were the reason for the objections voiced to the splitter type anti-vortex wall. The anti-vortex plate of Fig. 8(f) is stated to be equally efficient. It appears to be much simpler structurally, and should be a satisfactory replacement for the wall.

The second point concerns the limitations of any design which gives slug flow. Slug flow is unsteady flow and associated with such flow are varying pressures, noise, vibration, and pounding at the inlet, and intensified wave action at the outlet. It appears that acceptability or rejection of these flow conditions depends on the size of the works. For small stock-watering dams using an 18 in. or 24 in. pipe these problems are probably not critical. For large pipes of 60 in. or greater it would seem a wise precaution to avoid designs which give slug flow.

It should be mentioned that when the spillway operates, slug flow would prevail most of the time. This is because the inlet primes with a fairly low discharge, but the pipe does not flow full continuously until this discharge has been exceeded several times.

The final and most important point concerns the development of sub-atmospheric pressures in the inlet. It was stated under the heading "Theory"

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<sup>a</sup> May, 1960, by Fred W. Blaisdell.

<sup>10</sup> Assoc. Prof. of Civ. Engrg., Univ. of Saskatchewan, Saskatoon, Canada.

that if the pipe is steep, the pressure "will ordinarily be sub-atmospheric throughout most of the conduit length." This would also be true for any other type of inlet on a steep pipe flowing full. However, the local contraction of flow in the hood inlet will give a pressure much lower at that point than anywhere in the pipe.

Consider a sharp edged re-entrant tube flowing full: The coefficient of the jet contraction  $C_c = 0.5$ , and the coefficient of head loss

$$K_e = \left( \frac{1}{C_c} - 1 \right)^2 = 1.0.$$

The pressure head at the invert at the inlet will be

$$\frac{P}{w} = H - \frac{V^2}{2g} \dots \dots \dots (15)$$

where  $V$  is the local velocity of the contracted jet, and not the average velocity in the pipe. Therefore,

$$\frac{P}{w} = H - \frac{Q^2}{C_c^2 A^2 2g} \dots \dots \dots (16)$$

With  $C_c = 0.5$ , the local velocity head will be 4 times as great as the velocity head based on average pipe velocity, and the pressure head will be correspondingly lower. Eq. 16 has been verified experimentally by the writer and others for a sharp edged re-entrant tube flowing full.

Since the value of  $K_e$  for the hood inlet is also close to unity, one might expect that  $C_c$  would also equal 0.5. Actually, the cases are not exactly comparable, as the bevel cut on the hood inlet has some modifying influence. It is not possible to determine  $C_c$  for the hood inlet flowing as an orifice, because it will not flow as an orifice. However, the "effective" value of  $C_c$  can be calculated from Eq. 16 if  $Q$ ,  $A$ ,  $H$ , and  $P/w$  are measured.

In tests conducted by the writer, piezometer readings were taken along the pipe invert downstream from the hood inlet. The pressures were not as low for the hood inlet as for the sharp edged re-entrant. It was calculated that the effective value of  $C_c$  was approximately 0.6. This is a higher coefficient than for the ordinary sharp-edged re-entrant, but it appears reasonable for this case.

It was also observed that the crown pressures along the hood inlet were greater than 1 diameter above the invert pressures. This also appears reasonable when the upward component of velocity, directed toward the crown at the inlet, is considered. Actually, the flow pattern is exceedingly complex, and the value of  $C_c = 0.6$  is suggested only as a value for computing pressures, rather than giving an actual area of flow in the inlet.

Eq. 16 may be written in dimensionless terms by dividing by  $D$

$$\frac{P/w}{D} = \frac{H}{D} - \frac{\left( \frac{Q}{C_c A} \right)^2}{2gD} \dots \dots \dots (17)$$

Substitution of  $C_c = 0.6$ , and reduction, yields an equation in the same terms as Fig. 6:

$$\frac{P/w}{D} = \frac{H}{D} - 0.07 \left( \frac{Q}{D^{5/2}} \right)^2 \dots \dots \dots (18)$$



To illustrate the significance of Eq. 18, suppose a prototype pipe 48 in. in diameter were to be built with such a length, slope, and roughness as to yield the discharge curve shown in Fig. 6 for a pipe slope of 0.200. This pipe should flow full when  $H/D = 2$  and give a discharge corresponding to  $Q/D^{5/2} = 15.5$ . Substitution of these values in Eq. 18 shows that the pressure head at the invert would be -14.8 pipe diameters. This would correspond to -59.2 ft on the prototype, so obviously it would not perform as indicated on the model under these conditions.

A value of  $-1/2$  an atmosphere is often imposed as a design limit in hydraulic structures. Lower pressures are considered to be in the realm of potential cavitation. If this value was used in Eq. 18, then the permissible maximum size of pipe for the conditions of the above example would be 14 in.

In the writer's view, the negative pressures at the inlet impose a very definite limit on the size of pipe for which the hood inlet may be used with success. The designer should check each particular case, using Eq. 18, to insure that pressures are within acceptable limits.

## ERRATA

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 Journal of the Hydraulics Division

 Proceedings of the American Society of Civil Engineers
 

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April, 1960

p. 95. Delete lines 5 and 6 of paragraph 3 and substitute therefore: had proved, the dissipation of energy balances nearly with the production rate in the range of approximately  $0 < y/h < 1/2$ , and with the diffusion rate in the outer region ( $0.8 < y/h \leq 1$ ), and the turbulence

p. 95. Delete Eqs. 25 and 26 and replace with

$$\text{Dissipation} = A' \rho \nu \frac{\bar{u}^2}{\lambda^2} = A \rho \left( \frac{\sqrt{\bar{u}^2}}{\bar{U}_*} \right)^3 \frac{\bar{U}_*^3}{L} \dots\dots\dots (25)$$

$$\begin{aligned} \text{Diffusion} &= \frac{d}{dy} \left( \rho \mu \frac{u^2 + v^2 + w^2}{2} \right) = A'_1 \frac{d}{dy} \left( \rho \frac{u^2 v}{2} \right) \\ &= A'_1 \frac{\rho U_*^3}{h} \frac{d}{d(y/h)} \left( \frac{\bar{u}^2 v}{\bar{U}_*^3} \right) \dots\dots\dots (25') \end{aligned}$$

Considering the isotropic character of turbulence in the outer (open channel) or center (pipe) region, Eq. 25' can be written as

$$\text{Diffusion} = A_1 \frac{\rho \bar{U}_*^3}{h} \dots\dots\dots (25'')$$

Therefore, by equating Eq. 25 to Eq. 24 and Eq. 25'' we have

$$\frac{L}{h} = \left\{ \begin{array}{l} B \frac{y/h}{1 - y/h} \left( \frac{\sqrt{\bar{u}^2}}{\bar{U}_*} \right)^3 = f_1(y/h) \\ \hspace{10em} (0 < y/h < 0.5) \\ B' \left( \frac{\sqrt{\bar{u}^2}}{\bar{U}_*} \right)^3 = f'_1(y/h) \\ \hspace{10em} (0.8 < y/h \leq 1) \end{array} \right\} \dots\dots\dots (26)$$

p. 96. Just prior to the last paragraph insert

Also, the diffusion coefficient for uniform open channel flows is given from Eq. (28) as

$$D_z = \text{const. } \sqrt{g} h^{3/2} S_e^{1/2} \dots\dots\dots (31)$$

May, 1960

p. 21. In Eq. 9, insert a parenthesis before the term 0.15 and after the term  $Q/D^{5/2}$ .











# **PART 2**

AUGUST 1960 — 27  
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NO. HY 8  
PART 2

*Your attention is invited*

**NEWS  
OF THE  
HYDRAULICS  
DIVISION  
OF  
ASCE**



**JOURNAL OF THE HYDRAULICS DIVISION  
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS**



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## DIVISION ACTIVITIES

### HYDRAULICS DIVISION

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#### Proceedings of the American Society of Civil Engineers

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#### NEWS

August, 1960

#### THE PURPOSE OF THE HYDRAULICS DIVISION

"The advancement and dissemination of scientific and engineering knowledge in all branches of hydraulics, hydrology, hydraulic engineering and water resources. In particular this shall embrace meteorology and hydrology as the sciences dealing with the occurrence of water in the atmosphere, on the earth surface and in the ground, fluid mechanics for the understanding of all flow phenomena, applied hydraulics for the design and planning of hydraulic structures and of comprehensive systems, and those social, economic and administrative aspects basic to the conservation and utilization of water as an essential natural resource."

All committees and task forces together with their members are listed in the Official Register for 1960.

#### Committee Activity

##### Hydraulics Division Research Committee

At its meeting in Washington, D. C., on April 25 and 26, 1960, the Research Committee of the Hydraulics Division discussed ways and means of stimulating interest in Civil Engineering Research. One important action favored by the Committee is to raise the annual dues of the Society about \$5 and to reserve these additional funds for increasing research activities. Such serious action on the part of the Civil Engineering profession would indicate to the public and to prospective sponsors that research is important in the eyes of those most intimately acquainted with research needs. The effect of this would be to increase the availability of funds for research from both public and private sources.

The Committee discussed the problem of important unpublished data resting in files and how to get them published so they would become available for the use of the profession at large. The Committee is very anxious to help individuals having data published, and invites anyone who has or knows of such

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information to communicate with the Research Committee. If a description of the material is furnished with publication plans, if any, the Committee will offer its suggestions for publication or will refer the individual to an appropriate technical committee for assistance in evaluating the data and for recommendations or presentation or publication.

The Committee on Research of the Hydraulics Division is preparing an outline of eight typical basic research projects which may serve as examples of the kind of research which the committee feels should be undertaken by Civil Engineers.

The objective in preparing such material is to help people who have ideas for research to prepare proposals for research programs and to get financial support for them. It is the Committee's hope that these examples will help research workers to improve the presentation of their ideas so they will be accorded the best possible reception by prospective sponsors. Drafts of the outlines of these projects are being readied for circulation to experts in the fields covered for comments and suggestions. After revision they will be submitted to Civil Engineering for publication.

In undertaking this work the Research Committee is carrying out its assigned objective which in part is to initiate, organize, and coordinate programs of research in hydrology, hydromechanics and applied hydraulics, and to promote interest in and financial support for research in cooperation with the societies Committee on Research.

The following memorandum will be of particular interest to members of the Hydraulics Division:

#### "A SURVEY OF HYDRAULIC RESEARCH WORK AND ACTIVITIES

By R. D. Goodrich<sup>1</sup> and J. M. Robertson<sup>2</sup>

20 May 60

Following its reconstitution in 1957 as an administrative committee charged with the task of initiating, organizing, and coordinating hydraulic research, the Hydraulic Research Committee attempted to assess the hydraulic research activity and needs as currently visualized by the many committees of the Society. Since hydraulics not only includes certain application areas directly but also represents a subject basic to other fields encompassed within the work of several ASCE divisions, inquiries were directed both within and without the Hydraulics Division. The six technical committees of the Hydraulics Division and the research committees of the Engineering Mechanics (fluid Dynamics Comm.), Highway, Irrigation and Drainage, Pipeline, Sanitary Engineering, Soil Mechanics and Foundation (Comm. on Earth and Rock-fill Dams) and Waterways and Harbors Divisions were asked the following questions:

1. What research programs are now in progress under the direction, sponsorship, or prodding of your committee? (Include literature research and the formulation of new concepts based on elements of existing knowledge, as well as laboratory or field studies.)
2. What research programs do you consider should be undertaken within the area of responsibility of your committee?

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<sup>1</sup> Chairman 1958.

<sup>2</sup> Chairman 1959.



3. What plans do you have for getting the research programs you consider desirable underway?

Some very suggestive and pertinent information has been received, although some of the persons contacted took a narrow definition of research and ignored the parenthetical comments in Item 1. This memorandum attempts to summarize the ideas and information furnished. In line with the committee's responsibility to coordinate, this material should be given wide dissemination.

The survey of hydraulic research work has led to a picture of the research in hydraulics underway by or associated with the society, to some very valuable indications on how others are approaching the problems of promoting research in their areas, and to information on desirable hydraulics research projects. This summation on the information received will be presented in accord with this subdivision. In general the information on desirable projects came from the intra-divisional contacts but some came from the inter-divisional contacts.

#### ASCE Engendered Hydraulics Research

With the exception of the work reported by the Sanitary Engineering Research Committee, few of the committees contacted felt that they had any research in progress. Reference, however, to the assignments of various task forces under these committees strongly suggests that indeed research is underway. Utilizing the information given in the 1959 ASCE Official Register, an attempt has been made to circumvent this narrow interpretation of research. Presentation of the research underway by the Sanitary Engineering Research Committee is not included here since it is not hydraulics in the usual connotation—it will be discussed briefly in the next section. Due to a lack of familiarity with the efforts of the other division, this present summation is limited to the Hydraulics Division. In all probability other task force activities should, however, be included in this listing, e.g., Task Committee on Mechanics of Stratified Flow (Fluid Dynamics Committee of Engineering Mechanics Division).

The following task forces of the various technical committees of the Hydraulics Division appear to be concerned with at least a certain amount of hydraulic research activity:

- |  |   |                            |
|--|---|----------------------------|
| 1. Energy Dissipators for Spillways and Outlet Works | } | Hydraulic Structures Comm. |
| 2. Flow in Large Conduits                            |   |                            |
| 3. Hydrologic Data                                   | } | Hydrology Comm.            |
| 4. Spillway Design Floods                            |   |                            |
| 5. Aerated Flow in Open Channels                     | } | Hydromechanics Comm.       |
| 6. Cavitation in Hydraulic Structures                |   |                            |
| 7. Friction Factors in Open Channels                 |   |                            |
| 8. Local Scour                                       | } | Sedimentation Comm.        |
| 9. Sediment Distribution in Reservoirs               |   |                            |

All of these efforts fall into a certain class of research effort involving as they do a combination of literature research, compilation of available information in new forms, and encouraging interest in the several problems. These

are all desirable activities in important areas, however they do not represent all the hydraulics research that should be in progress.

#### Promotion of Research

A major problem facing this present committee is how to promote more of the needed hydraulic research. Some very valuable suggestions in this regard have been obtained from three sources, two of these from the inter-divisional contact letters and one from a less formal extra-society contact. In 1956 the Hydraulics Division of the ASME organized a Fluid Mechanics Sub-committee to encourage work in fundamental fluid mechanics. This committee has already made much headway. They first conducted a survey of the unsolved problems.\* One topic which stood out was stall or flow separation and its associated transient effects. This conclusion led directly to the second major effort holding two-day "stall symposium" at the ASME annual meeting in December 1958. Some 300 engineers attended, listened to formal presentations, gave informal comments and exchanged informal comments and information. It was obviously a valuable contribution to the research effort.

The committee on research of the Irrigation and Drainage Division has also followed a questionnaire approach. A response of about 53 per cent was received on 230 individuals contacted. The nature of the replies received led to the development of a list of 29 research studies needed urgently. Many of these will be included in the next section of this report. The Committee on Sanitary Engineering Research appears to have the most active program of research sponsorship underway of all those contacted within the ASCE. The committee has divided itself into seven separate sections pertaining to the various branches of sanitary engineering. Each section is headed by one man, an expert in his area, who is almost entirely on his own. The committee has taken two separate approaches to encouraging research. In the first, they attempted to locate, summarize and evaluate current research. This led to the submission of 18 research reports, which have been or are being published. The second approach (1957-1958) was the instigation of one new research project in each of the seven areas. Of the four upon which specific information was received, the support obtained, as of the spring 1958, varied from a grant from the National Institute of Health to contacts with the USPHS and APWA for possible funds.

#### Needed Hydraulics Research Studies

One of the obvious purposes of this present study was to obtain suggestions on hydraulic research work that is felt to be necessary. In this aspect the information received varied from ten complete research proposals (including estimated budget and time schedule) submitted by the Hydromechanics Committee as representative of the type of research worthy of support by the profession and the ASCE, to lists as obtained by the Research Committee of the Irrigation and Drainage Division. The various project studies needed will be listed or summarized in accordance with the source submitting them. A number of apparently non-hydraulic research needs have been omitted. No attempt has been made to eliminate duplications.

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\* S. J. Kline and R. C. Dean "Unresolved Problems of Fluid Mechanics," Mechanical Engineering, Dec. 1958, pp. 54-55.

## Hydromechanics Committee—

1. Diffusion in Flow Through Porous Media.  
Applications indicated in regard to mixing of waters of different quality or spread of pollution in a ground water basin.
2. Submerged Jet in a Flowing Stream (with and without density difference). Jet directed at an angle to main strain. Application in regard to discharge of sewage in river or ocean current or waste gas into wind.
3. Low Velocity Current Meter.  
Needed for laboratory model investigations and field measurements of harbor currents, ground water movement, etc.
4. Compact diffusers of Large Area Ratio.  
Application in sedimentation basins, desilting works, etc.  
  
(Note: work by Kline and associates at Stanford, appears to have this well in hand for "two-dimensional" diffusers. JMR.)
5. Critical Tractive Force of Sediments.  
Understanding of conditions under which sediments first start to move is fundamental to scour and erosion problems and sediment transportation problems.
6. Structure of Turbulence in Shear Flow.
7. Forces on Stilling Basin End Sills and Baffles.  
Systematic research study needed to measure the force on these to give the hydraulic engineer suitable design tools.
8. Discharge Coefficients for Flow Over Low Spillway Crests.  
Need to determine the discharge coefficient for flow over a properly shaped crest at less than design head, since low crests often used as controls for chute spillways and designed without regard to change in shape for appreciable approach velocities.
9. Tidal Phenomena in Estuaries and Bays.  
Better understanding of tidal phenomena is needed; there is almost no experimental confirmation of basic tidal theories. This would be a systematic laboratory study of the separate effects of estuary geometry and boundary resistance for damped oscillating tides.
10. Withdrawal of Fluid from Systems with Density Gradients.  
Application in the selective withdrawal of water from water having thermal (density) gradients, as for cooling water in steam power plants.
11. Spillway Design Research Needs.  
Detailed outline of all facets of spillways with indications of research problems (some 70) prepared by F. B. Campbell, November, 1956.

## Hydraulics Structures—

12. Spreading of Jets on a Sloping Floor.
13. Wave Pressures on Spillway Crest Gates.
14. Losses at Bifurcations and Trifurcations (diverging flow) in Large Conduits.
15. Hydraulic Characteristics of Reverse Tainter Valves.
16. Generalized Equations for Discharge of Tainter Gates.
17. Allowable Irregularities in Hydraulic Surfaces which can be Tolerated without Producing Cavitation.

(Note: committee was evidently not aware of Navy sponsored work on this subject which has recently been reported (cf J. W. Holl, ASME paper 59-Hyd-12). Application of this to problems considered should be reviewed before further work is considered. JMR.)

#### Flood Control—

18. Runoff from Small Watersheds.
19. Methods of Computing Backwater Curves by Electronic Computing Machines.
20. Assembling Up-to-Date Information on Manning's n.
21. A Study of Correlation of Floods with Meteorological Events.

#### Hydrology—

22. Groundwater Studies, including movement of saline waters, analysis and interpretation of fluctuations, etc.
23. Peak Runoff Studies of Small Watersheds.
24. Extension of Current Procedure for Estimating Synthetic Unit Hydrographs.
25. Investigations of Cyclical Trends in Meteorologic and Hydrologic Phenomena.

#### Irrigation and Drainage Division—Committee on Research

26. Methods of Reducing Reservoir Evaporation.
27. Improved Techniques for Recharging Aquifers.
28. Development of Design or Arrangement Criteria for Diversion Dams and Headgates to Reduce Sediment Entry.
29. Development of Simple Design Criteria for Roughness in Lined and Unlined Channels.
30. Hydraulics of Surface Methods of Irrigation.
31. Development of Improved Methods for Measuring Irrigation Intake Rates.
32. Development of Design Criteria for Open-Drain Bank Stabilization.
33. Development of Accurate, Low Cost Means for Measurement of Low Flows in Channels.
34. Criteria for Design of Stable Channels.
35. Criteria for Designer of Excluders and Ejectors for Sediment Control.
36. Laboratory and Field Studies of Weather Control.

#### Waterways and Harbors Division—Committee on Research

Suggested list of Research Problems Related to Waterways and Harbors Appropriate for Study in Smaller Private Laboratories.

37. Stream Channel Problems—Flood Channel Deterioration.  
Model scale study of factors causing meanders, etc.
38. Stream Channel Problems—Bank Erosion by Wave Action.  
Besides mechanism of erosion processes, need information on wave character from ships, small boats and local winds.
39. Sediment Supply to Shorelines by Streams.
40. Movement of Sediment by Wave Action.
41. Mechanics of Sediment Transport in Littoral Zone.
42. Characteristics of Waves Occurring.

43. Standing Waves and Seiches (Harbor Surging).
44. Dynamic Wave Forces.  
Type of force, structure and characteristics of structure.
45. Ship Waves in Shallow Water.
46. Wind Tides.
47. Functional Design Methods for Shore protection Structures Jetties, Breakwaters, Groins, Bulkheads, etc.
48. Erosion and Scour of Pier Foundations.
49. Sources of Estuarine Sediments.
50. Salt Water Intrusion and Salt Water Wedge Phenomena.
51. Better Sediment Sampler for Use in Estuaries.
52. Improved Current Meter.  
More sensitive and indicate direction as well as current.

The above list of research needs is far from complete and the committee could continue to devote its major attention to extending and improving the list. However, the committee feels that the important job to be done is to stimulate research into these important problems and that the mere collection of lists of needed research is of little value in itself unless some action is taken. Years ago the old Hydraulics Research Committee compiled a carefully considered list of needed research. Reference to that list indicates little progress toward really solving any of the problems. This committee could add more items to the list. What is more important, however, is to note which of the problems appears again and again in various lists—i.e., which are of paramount importance, such as in the case of stall for the ASME Hydraulic Division Sub-committee on Fluid Mechanics. When a few such problems, in their fundamental or basic aspects, are singled out then strenuous efforts should be applied to getting them researched. This is roughly the procedure of the Sanitary Engineering Research Committee. A good start toward a list of a few fundamental problems has been furnished by the Hydromechanics Committee."

#### Members Active in International Meeting

The 12 General Assembly of the International Union of Geodesy and Geophysics was held in Helsinki, Finland, on July 25 to August 6, 1960. Some 45 papers on hydrology were presented by American authors in the sessions of the International Association of Scientific Hydrology. ASCE was well represented with nearly one-third of the hydrology papers being authored by Society members. Following is a list of ASCE members participating and their papers:

1. Lindner, C. P., "Currents in Tidal Reaches of Rivers and Their Effect on Shoaling of Side Basins."
2. Linsley, R. K., Jr. (with Crawford, N. H.), "Computation of a Synthetic Streamflow Record on a Digital Computer."
3. Langbein, W. B., "Water Levels as Indicators of Long-Term Precipitation and Runoff."
4. Walton, W. C. (with Neill, J. C.), "Analyzing Ground Water Problems with Mathematical Models and a Digital Computer."
5. Rorabaugh, M. I., "Use of Water Levels in Estimating Aquifer Constants in a Finite Aquifer."



6. Simpson, E. S. (with Lieberman, J. A.), "Practices and Problems in Disposal of Radioactive Wastes into the Ground."
7. Henry, H. R., "Salt Intrusion into Coastal Aquifers."
8. Todd, D. K., "Salt Water Intrusion of Coastal Aquifers in the United States."
9. Snyder, F. F., "Evaporation on the Great Lakes."
10. Maddock, Thomas, Jr., "Erosion Control on Five Mile Creek, Wyoming."
11. Peterson, H. V. (with Hadley, R. F.), "Effectiveness of Erosion Abatement Practices on Semiarid Rangelands in Western United States."
12. Light, Phillip, "Snow Melt Floods, Spring 1959, Upper Mississippi Watershed."
13. Case, J. B., "Glacier-Mapping Activities in the United States."

#### Newsletter Read?

The Editor has wondered many times concerning the interest of the members in Newsletter items. Carl Kindsvater reports that he has already received a hundred requests for his draft on weir standards noted in the April 1960 issue of the Newsletter. These requests have come from hydraulics men in many parts of the world. It is encouraging to your Editor to know that the Newsletter is read by so many.

#### BOSTON ASCE CONVENTION

October 10-14, 1960

The Boston area should have its interesting fall characteristics on display. Plan to attend this meeting and enjoy the historical sights, East Coast beauty, and the five interesting sessions of the Hydraulics Division.

One of the many interesting social activities will be the Open House at the Hydrodynamics Laboratory, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, Tuesday, October 11, 1960, from 8 to 9:30 p.m.

#### Technical Program

##### Session on Tidal Hydraulics:

Presiding: Arthur T. Ippen, Chairman, Executive Committee; and Raymond Boucher, Member, Tidal Hydraulics Committee.

##### Estuarial Sediment Transport Patterns

H. A. Einstein, Professor of Hydraulic Engineering, Department of Civil Engineering, University of California, and R. B. Krone, Assistant Research Engineer, Sanitary Engineering Research Laboratory, University of California, Berkeley, California.

##### The Analysis of Salinity Intrusions in Well Mixed Tidal Estuaries

Arthur T. Ippen, Professor of Hydraulics, Massachusetts Institute of Technology, and D. R. F. Harleman, Associate Professor of Hydraulics, Massachusetts Institute of Technology, Cambridge, Massachusetts.

##### Experimental Results on Salinity Intrusions in Estuaries

Henry B. Simmonds, Chief, Estuaries Section, Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi.



The Tide in the St. Lawrence River

H. A. Neu, Hydraulics Laboratory, Division of Mechanical Engineering, National Research Council, Ottawa, Ontario, Canada.

## Session on Hydrology:

Presiding: Arthur T. Ippen, Chairman, Executive Committee; and a member of the Hydrology Committee.

Estimating Potential Evapotranspiration

W. Russell Hamon, Research Associate, Travelers Insurance Co., Hartford, Connecticut.

Use of High Speed Analog Computer for Routing of Storm Rainfall Through a River Basin

Henry M. Paynter, Assoc. Prof. of Mechanical Engineering, Mass. Institute of Technology, Cambridge, Massachusetts.

Infiltration of Fluids in Dry Porous Media

A. I. Johnson and W. N. Palmquist, Hydrologic Laboratory, U. S. Geological Survey, Denver, Colorado.

Use of Digital Computers in Computing Flow Hydrographs from a Network of Sub-Areas

W. L. Northrop and C. W. Timberman, Corps of Engineers, Kansas City District, Kansas City, Missouri.

## Session on Sedimentation:

Presiding: Arthur T. Ippen, Chairman, Executive Committee; and Robert B. Banks, Member, Sedimentation Comm.

Research Needs in Sedimentation

H. A. Einstein, Professor of Hydraulic Engineering, Department of Civil Engineering, University of California, Berkeley, California.

Effect of Bridge Constrictions on Scour

H. K. Liu, Assist. Prof., Colorado State University, Fort Collins, Colorado, and F. M. Chang, Research Assistant, Colorado State University, Fort Collins, Colorado.

Sediment Distribution in SCS Flood Water Retarding Structures

H. G. Heinemann, Hydr. Engr., Soil and Water Conservation Research Division, ARS, Lincoln, Nebraska.

The Mechanics of Washout of an Erodible Fuse Plug

E. Roy Tinney, Head, R. L. Albrook Hydraulic Laboratory, Washington State University, Pullman, Washington, and H. Y. Hsu, International Engineering Co., San Francisco, California.

## Session on Hydromechanics:

Presiding: Arthur T. Ippen, Chairman, Executive Committee; and a member of the Hydromechanics Committee.

Anisotropic Turbulence and Wall Effects in Open Channel Flow

Paul G. Mayer, Associate Professor of Civil Engineering, Georgia Institute of Technology, Atlanta, Georgia.

The Flow of Sand-Water Mixtures in Horizontal Pipes

Dr. Norbert L. Ackermann, Professor of Civil Engineering, University of Khartoum, Sudan, Africa, and Thomas E. Stelson, Head, Department of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, Pennsylvania.

Effect of Roughness, Spacing in Rigid Open Channels

William W. Sayre, Assistant Civil Engineer, Department of Civil Engineering, and Maurice L. Albertson, Director, Research Foundation, Colorado State University, Fort Collins, Colorado.

Session on Hydraulic Structures:

Presiding: Arthur T. Ippen, Chairman, Executive Committee; and H. J. Tracy, Hydr. Engr., USGS, Atlanta, Georgia.

The Effect of Spur Dikes on Flood Flows through Bridge Constrictions

John B. Herbach, Department of Civil Engineering, Lehigh University, Bethlehem, Pennsylvania.

Hydraulics of River Flow under Arch Bridges

J. W. Delleur, School of Civil Engineering, Purdue University.

Flow Through Multi-Opening Constrictions

John Shen, U. S. Geological Survey, Washington, D. C.

Note on the Ninth IAHR Convention

The subjects for this convention which will be held September 3-7, 1961, have been chosen. They are as follows:

1. The effect of turbulence on hydraulic structures.
2. Mechanics of ground water flow.
3. Hydraulic problems for computers.
4. Modifications of natural streams by engineering structures.

In addition, there will be two seminars:

1. Hydraulics of small structures in irrigation and drainage.
2. Fundamental hydrodynamics of free surface flow and of unsteady flow.

For Your CalendarASCE Meetings

October 10-14, 1960  
April 10-14, 1961  
August 16-18, 1961

October 16-20, 1961  
February 1962  
May 1962  
October 15-19, 1962

ASCE, Boston Convention  
ASCE, Phoenix Convention  
Hydraulics Division Conference,  
Urbana, Illinois  
ASCE, New York Convention  
ASCE, Houston Convention  
ASCE, Omaha Convention  
ASCE, Detroit Convention

Non-ASCE Meetings

September 1960	Hydraulic Turbine Research Symposium, IAHR, Nice, France
September 5-10, 1960	Conference on Hydraulics (River Hydraulics and Hydraulics of Infiltration) Hungarian Academy of Sciences, Budapest, Hungary
October 29-November 2, 1960	Annual Meeting Geological Society of America, Division of Engineering Geology, Denver, Colorado
June 26-July 2, 1961	Seventh Congress International Committee on Large Dams, Rome, Italy
September 3-7, 1961	Ninth IAHR Convention, Belgrade, Yugoslavia
September 6-8, 1961	Seventh Midwestern Conference on Fluid Mechanics and Soil Mechanics, Michigan State University, East Lansing, Michigan
June 18-21, 1962	Fourth National Congress of Applied Mechanics, University of California, Berkeley, California

## Use of the Hydraulics Division Newsletter

You are urged to continue use of the Division Newsletter for announcements, inquiries, personnel news, committee reports, surveys and other items of interest to Division members. A short note summarizing the highlights of committee meetings is particularly requested. Suggestions for improvement of the Newsletter will be appreciated. All contributions are appreciated.

Deadline dates for Newsletter contributions: October 1960 issue—August 20; December 1960 issue—October 20.

## COMBINED INDEX TO ASCE PUBLICATIONS

For complete coverage of the Society's 1959 year in print, there is now a Combined Index covering the Division Journals, Transactions, and Civil Engineering. Also included are reprints of the Proceedings Abstracts that are published each month in Civil Engineering. The price of the Combined Index

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August, 1960

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James W. Ball, Editor  
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